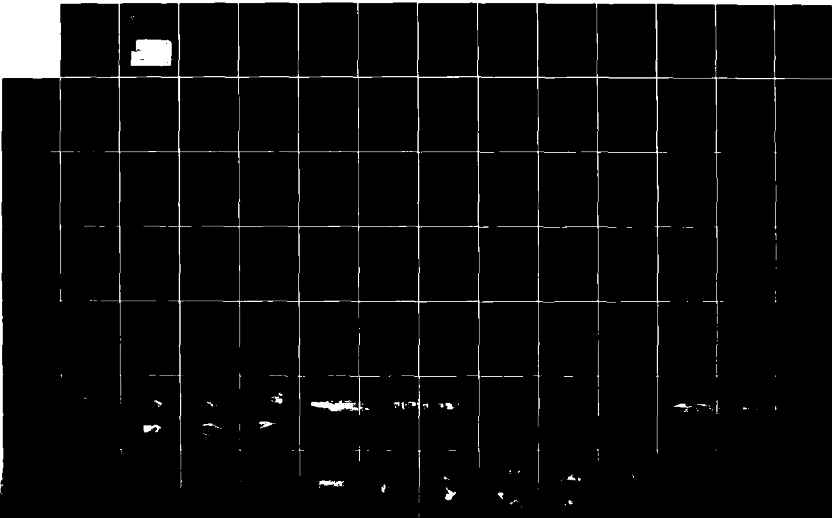
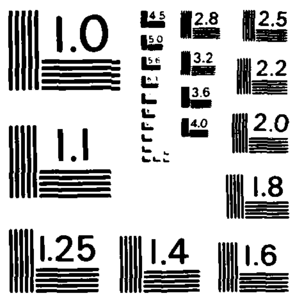
AD-A140 987 GRANGER LAKE EMBANKMENT-OUTLET WORKS-SPILLWAY VOLUME 1 1/2
(U) ARMY ENGINEER DISTRICT FORT WORTH TX G M RUEDE
AUG 83

UNCLASSIFIED F/G 13/2 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

US Army Corps
of Engineers
Fort Worth District

GRANGER LAKE

FINAL FOUNDATION REPORT

EMBANKMENT-OUTLET WORKS-SPILLWAY

VOLUME-1
AUGUST 1983

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER	2. GOVT ACCESSION NO. AD-A140 987	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Final foundation report, Granger Lake, embankment-outlet works-spillway		5. TYPE OF REPORT & PERIOD COVERED Final report 11/72 - 12/79
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) George M. Ruede, Project Geologist		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS U.S. Army Corps of Engineers, Fort Worth P.O. Box 17300 Fort Worth, Texas 76102		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS U.S. Army Corps of Engineers, Fort Worth P.O. Box 17300 Fort Worth, Texas 76102		12. REPORT DATE August 1983
		13. NUMBER OF PAGES 2 volumes
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report)
		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Embankment
Foundation
Granger Dam
Granger Lake
Outlet works
Spillway

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Granger Dam and Lake, called Laneport Dam and Lake until 3 January 1975, is located in central Texas about 6.5 miles east of Granger, Texas and 9.5 miles northeast of the city of Taylor, Texas. The structure is on the San Gabriel River, 31.9 river miles upstream from its confluence with the Little River. Principal structures of the dam are: a) earthfill embankment approximately 15,240 feet long, maximum height of 114 feet, top elevation of 555 feet, and crown of 30 feet; b) gate controlled outlet works; and c) spillway 950 feet wide, with a crest elevation of 528 feet, located on the right (south) abutment.

This final foundation report records, documents and provides solutions to problems with regard to foundation conditions (during the construction phase), problems encountered and methods/solutions to resolve foundation problems during actual construction of Granger Dam and Lake. Additionally this report provides information and suggested changes in design of dams (structures) on similar foundations.

CORPS OF ENGINEERS

FORT WORTH DISTRICT, TEXAS

FINAL

FOUNDATION REPORT

GRANGER LAKE

EMBANKMENT - OUTLET WORKS - SPILLWAY

-BY-

GEORGE M. RUEDE

PROJECT GEOLOGIST

FORT WORTH, TEXAS

AUGUST 1983

PREFACE

This report was prepared in the Geology Section of the Geotechnical Branch, Engineering Division, Fort Worth District. The text was written by George M. Ruede, Granger Project Geologist, under the supervision of Melvin G. Green, Chief of the Geology Section, and Wayne E. McIntosh, Chief of the Geotechnical Branch. Foundation approval and mapping were done by the author and Messrs. Raymond T. Hagen and Alan Marr, geologists. The author (in part) and a number of engineering interns, technicians, and draftsmen compiled the accompanying maps on the foundation from stadia survey data, quantity profiles, plane table sheets, and notes.

TABLE OF CONTENTS
VOLUME 1

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

CONSTRUCTION PHOTOGRAPHS

TABLE OF CONTENTS (continued)

CONSTRUCTION PHOTOGRAPHS

TABLE OF CONTENTS (continued)

CONSTRUCTION PHOTOGRAPHS

TABLE OF CONTENTS (continued)

CONSTRUCTION PHOTOGRAPHS

TABLE OF CONTENTS (continued)

CONSTRUCTION PHOTOGRAPHS

TABLE OF CONTENTS (continued)

CONSTRUCTION PHOTOGRAPHS

TABLE OF CONTENTS (continued)

CONSTRUCTION PHOTOGRAPHS

TABLE OF CONTENTS (continued)

CONSTRUCTION PHOTOGRAPHS

TABLE OF CONTENTS
VOLUME 2

PLATES

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

GRANGER FOUNDATION REPORT

INTRODUCTION

1. Project Location and Description. Granger Dam and Lake, called Laneport Dam and Lake until 3 January 1975, is located in central Texas about 6.5 miles east of the city of Granger, Texas, and 9.5 miles northeast of the city of Taylor, Texas. See plate No. 1 for the location map. The dam is located on the San Gabriel River, 31.9 river miles upstream from its confluence with the Little River. The principal structures of the dam are:

a. An earthfill embankment approximately 15,240 feet long, excluding the spillway, which has a maximum height of 114 feet, a top elevation of 555 feet, and a crown width of 30 feet.

b. A cut-and-cover, gate controlled outlet works, the conduit having an inside diameter of 18 feet. The gate-control tower has two 8 feet 6 inch by 19 feet 0 inch, hydraulically operated gates with invert elevations of 457 feet. There are also four 3 foot by 4-foot gates with inverts of 486.0 feet, 492.0 feet, 498.0 feet, and 504.0 feet. The height of the gate control tower is 102 feet above the invert (to top of parapet wall).

c. A concrete, uncontrolled ogee spillway 950 feet wide, with a crest elevation of 528 feet, located on the right (south) abutment.

2. Construction Authority. Congressional authority for the construction of Laneport Reservoir was originally contained in the Flood Control Act approved 3 September 1954 (Public Law 780, 83d Congress, 2d Session) in accordance with the plan of improvement as outlined in House Document No. 535 (81st Congress, 2d Session). A congressional resolution, adopted 29 July 1955, requested that House Document No. 535 be reviewed to determine if a change in the site of Laneport Reservoir was advisable. The Flood Control Act approved 23 October 1962 (Public Law 874, 87th Congress, 2d Session) authorized the construction and operation of North Fork and South Fork Reservoirs in conjunction with the authorized Laneport Reservoir as outlined in House Document No. 591 (87th Congress, 2d Session). Authority to initiate advance planning is contained in the Public Works Appropriation Act of 1965, approved 30 August 1964 (Public Law 88-511) and in Advice of Allotment C-124 dated 9 September 1964. In accordance with Public Law 93-631, dated 3 January 1975, Laneport was renamed "Granger Dam and Lake."

3. Purposes of Report. This report has been prepared pursuant to Regulation No. 1110-1-1801 to record foundation conditions during construction. The report is also intended to record unanticipated foundation problems encountered during construction and methods used to overcome them. The report includes suggested changes in design of dams on similar foundations.

4. Project History. As conceived and described in the original project documents, the dam was to be located at river mile 29.7 on the San Gabriel River and was to consist of an earthfill embankment and a controlled spillway on the right abutment. Subsequently, the dam was resited upstream at river mile 31.9 to obtain better foundation conditions and to take advantage of topography that would allow a shorter, less costly embankment. Design Memorandum No. 4 (Laneport), General, was submitted in January 1967 for the new site at river mile 31.9. The design consisted of a controlled spillway (tainter gates) on the right abutment, with vertical and horizontal drainage blankets in the embankment.

Supplement No. 1 to Design Memorandum No. 4 was submitted in May 1967, proposing a controlled spillway located in the right side of the flood plain instead of on the right abutmment. A meeting of the Board of Consultants for the project was held in November 1967 to review the design. The Board agreed with the general embankment design, but recommended elimination of the vertical and horizontal drainage blankets in the embankment. The Board also requested that a study be made comparing an uncontrolled spillway on the right abutment and a flood plain outlet works with the controlled flood plain spillway.

Supplement No. 2 to Design Memorandum No. 4, submitted in January 1968, presented project formulation data which included comparisons of water storages, yields, benefits, and costs for various sized projects and order of construction based on water resources of the San Gabriel River watershed.

The Board of Consultants met a second time in May 1968. The Board and representatives of OCE and SWD selected the combination of an uncontrolled spillway on the right abutment and a cut-and-cover outlet works in the valley, but recommended that a study be made to compare a tunnel through the right abutment with the cut-and-cover outlet works. The Board met a third time in December 1968, primarily to select the location and type of outlet works to be used. The cut-and-cover outlet works was selected. This plan was presented in Supplement No. 3 (Revised) to Design Memorandum No. 4, General, submitted in July 1969. In addition, Supplement No. 3 contained soils design data and stability analyses governing design of the embankment, which had not been presented previously.

Supplement No. 4 to Design Memorandum No. 4, submitted in January 1971, contained replies to comments and minor revisions in accordance with endorsements to Supplement No. 3 (Revised).

Supplement No. 5 to Design Memorandum No. 4, submitted in October 1976, presented changes in the spillway design to result in an expected saving of $4,781,000. The principal change was to excavate completely the low shear strength, weathered Shale 1 (see discussion of geology) from the spillway and replace it with a gravel refill obtained from

required spillway excavation. An impervious clay refill was to be employed upstream from the key under the "A" slab lane to prevent seepage from reaching the gravel refill downstream. This design change founded the approach slab, the ogee weir, and the upper portion of the chute slab, together with their walls, on refill. Because of the greater shear strength of the gravel refill, the chute was shortened approximately 139 feet from the earlier design. Anchors and keys under the wall footings were eliminated in the refill area. This design change was accepted for construction. The term refill, as used here and in the following text, means the extensive compacted fill used as a substitute for a foundation of weathered shale.

5. Location of Structures.

a. Embankment. The embankment commences immediately north of Farm-to-Market Road 1331 on the south or right abutment at approximate station 1+85 and extends north through a 6 degree left curve to the right spillway wall at station 14+56.69. From the left spillway wall at station 24+10.69, the embankment extends northward through an 8 degree right curve to station 30+89.75. From here it crosses the San Gabriel valley, uncurving, to terminate at station 163+75 on the left abutment near relocated Farm-to-Market Road 971. The embankment crosses the San Gabriel River at station 62+10, at an angle of about 60 degrees to the course of the flood plain, Willis Creek at station 79+60, Sorefinger Creek at station 108+60, and an unnamed tributary to Sorefinger Creek at station 118+40. The azimuth of the centerline of the embankment in the valley is N 22° 42.6' E (true)

b. Outlet Works. The outlet works is situated in a terrace north of the flood plain, the ground surface of which is about 35 feet above the flood plain. The outlet works crosses the centerline of the embankment at outlet station 10+00, which is station 94+10 of the embankment. The conduit extends 365 feet upstream and 510 feet downstream from the centerline of the embankment. The gate control tower, including the transition to the conduit, extends 95 feet upstream from the conduit. The approach slab and walls extend an additional 131 feet upstream from the gate control tower, or a total of 591 feet from the centerline of the embankment. The chute and stilling basin extend 225 feet downstream from the conduit, or a total of 735 feet downstream from the centerline of the embankment.

c. Spillway. The spillway is located on the right abutment, its centerline crossing the centerline of the embankment alignment at spillway station 29+16.27 and embankment station 19+33.69. The approach walls extend 120 feet upstream from the embankment alignment, but the approach slab extends only 30 feet upstream. The crest of the ogee weir is located 50 feet downstream from the trace of the embankment centerline; the bottom of the chute is 294.78 feet downstream, and the end of the stilling basin is 399.0 feet downstream. The training walls terminate 20 feet beyond the end of the stilling basin.

6. Contracts, Contractors. Granger dam was constructed under three contracts. The initial contract was for construction of sections of the embankment from approximate station 1+85 to approximate station 12+10 and from station 26+40 to station 60+60, and for partial excavation of the spillway, the excavated materials from which were used in the embankment. The second contract was for construction of the outlet works. Materials excavated from the outlet works were used to construct an upstream section of the embankment involving the random and semi-compacted zones. The third contract was for completion of the embankment and construction of the spillway. Contract data follow.

a. Initial Embankment and Partial Excavation of the Spillway.

Contract No.: DACW63-73-C-0040
Contractor: Dahlstrom Corporation, Dallas, TX
Bid: $3,141,677.20
Notice to proceed: 13 November 1972
Work commenced: 13 November 1972
Work completed: 1 June 1975
Total payment: $3,326,101.05 (after 167-day late penalty)

b. Outlet Works.

Contract No.: DACW63-74-C-0047
Contractor: Hensel Phelps Construction Company, Greeley, CO
Bid: $4,436,000
Notice to proceed: 7 January 1974
Work commenced: 14 January 1974
Work completed: 30 April 1976
Total payment: $5,862,214.78

Subcontractor, earthwork, excavation: Udo Haufler Company, Austin, TX
Subcontractor, earthwork, backfill: Spiller Construction Company, Wimberley, TX

c. Completion of Embankment and Spillway, and Access Roads and Service Bridge.

Contract No.: DACW63-76-C-0057
Contractor: J. D. Abrams Company, El Paso, TX
Bid: $18,682,343.60
Notice to proceed: 17 February 1976
Work commenced: 23 February 1976
Work completed: 18 December 1979
Total payment: $22,000,962.25 (after claim)

Subcontractor, concrete construction: Clearwater Constructors, Austin, TX (subsidiary of Hensel Phelps Construction Company, Greeley, CO)

7. Resident Construction Staff. Supervision of construction under the first contract, Initial Embankment and Partial Spillway, was directed by the Fort Hood Resident Office, Fort Hood, Texas. Field inspection was administered from the San Gabriel Project Office, Georgetown, Texas. Resident personnel were:

Fort Hood Resident Engineer - Arthur F. Brown
San Gabriel Project Engineer - Ronald P. Zunker
Office Engineer - Robert E. Beggs
Construction Representative (Soils) - James C. Mebane
Construction Representative (Laboratory) - Ray M. Gordon
Engineering Technician - Jack Syers
Engineering Technician - Lawrence L. Mann
Engineering Trainee - Mark W. Whiteley

Prior to commencement of the second contract, Outlet Works, the Fort Hood Resident Office had been redesignated the Central Texas Area Office and the San Gabriel Project Office had become the San Gabriel Resident Office. Resident personnel supervising construction were:

Central Texas Area Engineer - Arthur F. Brown
San Gabriel Resident Engineer - Ronald P. Zunker
Project Engineer (briefly) - Bradford H. Craig
Construction Representative (Soils and Concrete) - James C. Mebane
Construction Representative (Laboratory) - Ray M. Gordon
Construction Representative (Mechanical and Electrical) - A. L. Paul
Civil Engineer - Mark W. Whiteley

Supervision of construction under the third contract, Completion of Embankment and Spillway, was directed initially from the Central Texas Area Office. Late in the period of construction, direction was transferred to the North Texas Area Office at Fort Worth, Texas, which was later moved to Georgetown, Texas. Resident personnel supervising construction were:

Central Texas Area Engineer - Arthur F. Brown
San Gabriel Resident Engineer - Ronald P. Zunker
North Texas Area Engineer - Ronald P. Zunker
Granger Project Engineer - Eugene L. McDaniel
Construction Representative (Soils) - James C. Mebane
Construction Representative (Concrete) - A. L. Paul
Construction Representative (Laboratory) - Ray M. Gordon
Laboratory Technician - Randell J. Cosper
Various trainees and laboratory technicians for short periods of time.

FOUNDATION EXPLORATIONS

1. Investigations Prior to Construction. Since only a very small amount of the foundation bedrock could be viewed or sampled at the ground surface, the foundation was explored mainly by borings, pits, and trenches. Initial foundation exploration began in 1946 at the original site at river mile 29.7 by drilling six combination fishtail/2-inch core borings. In 1965, a total of 27 additional borings of varying types were drilled on this site. A total of 126 borings (auger, Denison sampler, and core), including five 30-inch or 36-inch diameter auger borings, were drilled at the project site at river mile 31.9 prior to construction. This number does not include borrow investigations. Geophysical logs were made in most of these borings, usually after they were drilled deeper with a fishtail rotary bit, to provide information on the deeper stratigraphic horizons of the foundation. Additional exploration of structure of the bedrock was done by geophysically logging 78 fishtail borings, drilled solely for that purpose. Five specially designed wells were completed to test for perched water conditions at a variety of locations across the project. Also, three test pits and two inspection trenches were excavated for direct observation of the character of the foundation overburden and rock.

Four water wells, intended for temporary use as a water supply for construction and operation of the project office building, were completed in the overburden sand and gravel between May and November 1972 in the dismantled town of Friendship, located near the left abutment. One of the wells was abandoned as nonproductive immediately after completion. The three remaining wells were ultimately abandoned because of either bacterial contamination or insufficient yield. The project office was supplied by hauling water during construction.

2. Investigations During Construction. In late July and August 1974, eight test pits, approximately 215 feet of backhoe trench, and 27 shallow 6-inch core borings were employed to reexplore the foundation of the conduit, gate tower, and approach structure of the outlet works to provide data for redesign of foundation grades to eliminate the lower depositional seam (see discussion in paragraphs 2c and 4b following). A total of 11 percolation tests were performed in Friendship and Wilson H. Fox Parks in December 1978.

GEOLOGY

1. Regional. The region surrounding Granger Dam is situated on the outcrop belts of several formations of upper Cretaceous age. (The term region is used in an areally restricted sense here.) These outcrops strike approximately 20 degrees east of north, and the formations involved dip eastwardly at an average rate of approximately 67 feet per mile. The dip and strike of the bedrock formations affect the courses of the streams of the region in that most of the streams flow eastwardly, approximately in the direction of formational dip. These streams join Brushy Creek which flows northeastward to its confluence with the Little River along the outcrop belt of the Midway Group of lower Tertiary geologic age. The Little River turns abruptly northeastward at its confluence with Brushy Creek so as to appear to be a 10-mile extension of Brushy Creek. Those upper Cretaceous formations which comprise the regional bedrock are, from oldest (lowest) to youngest (uppermost): the Austin Formation of limestone ("chalk") 325 feet to 420 feet thick; the Taylor Group (composed of the basal Ozan Formation of shale 300 to 400 feet thick; the Pecan Gap Formation of very calcareous shale 50 to 200 feet thick; and the Marlbrook Formation of shale of indeterminate thickness); and the Navarro Group, also primarily composed of shale of indeterminate thickness. The Marlbrook Formation and Navarro Group, lithologically indistinguishable from each other, have an aggregate thickness of about 600 feet. The Midway Group overlies the Navarro Group. Between the city of Taylor, south of Granger Dam, and the Little River, north of the dam, The Texas Bureau of Economic Geology maps all three members of the Taylor Group together with the whole of the Navarro Group as a single undifferentiated unit[1]. Beyond these places, the Ozan and Pecan Gap Formations are separately mapped, but the Marlbrook Formation and the Navarro Group are mapped as a single unit.

The dominant structural feature of this region, exclusive of regional dip, is the Balcones trend of faulting. Faults of the Balcones system are extension phenomena, with down-to-the-east fault block movement. The Balcones system is concentrated in a narrow strip a little more than 4 miles wide at Austin, Texas, where faults have large, vertical displacements. This strip broadens considerably to as much as 20 miles just north of Austin, where it enters our region of interest. As it broadens, faulting tends to be confined to three principal trends, (as mapped on the Geologic Atlas of Texas by the Texas Bureau of Economic Geology, Austin Sheet). Balcones faults generally strike from about north 30° east to north 40° east, but may strike locally as little

---

[1]GEOLOGIC ATLAS OF TEXAS, AUSTIN SHEET 1974, The University of Texas at Austin, Bureau of Economic Geology.

as north 20° east. The fault trend nearest to Granger Lake, as mapped, lies 2.5 miles west of the town of Granger, Texas, near the upper end of the lake. Another trend of mapped down-to-the-east faulting (mapped separately from the Balcones system) extends from south of Bastrop, Texas, northeastward to within a few miles of Hearne, Texas. Faults of this trend also strike from 30° to 40° east of north. A number of Balcones type faults are known by drilling to exist between the mapped Balcones fault trend west of Granger, Texas, and the mapped fault trend to the east described above.

Bedrock in most of the divide areas between major streams of this region is extensively mantled by thick upland terrace deposits. These deposits typically are comprised of a basal gravel, then sand or sandy clay, and a surface layer of clay. Locally, the basal gravel is commercially exploited. A few terrace deposits are present along some of the streams of the region. These terraces tend to occur 30 to 50 feet above the elevation of the nearby flood plain. The constituent materials of the local terrace deposits are essentially the same as those of the upland terraces, except that the basal gravel is usually thinner. The flood plain of the Little River and those of the several streams flowing eastwardly to Brushy Creek are typically comprised of a sequence of materials very like those of the terraces. Elsewhere, overburden mantling the bedrock is usually either residual soil generated by extreme weathering of the bedrock beneath, sheet wash deposits, or alluvium of minor tributaries. Usually the residual soils of this region are clay because of the preponderance of shale bedrock. Volumetrically, the gravel of the terraces and flood plains are the only significant source of shallow ground water (where water table conditions prevail). The upland terraces in the divides between the streams contain the greatest quantity of usable ground water because of their great areal extent and relatively greater thicknesses of gravel and sand.

2. Site Geology.

a. Physiography. The embankment from its right (south) end to the spillway, the spillway proper, and the embankment from the spillway to about station 35+00 are situated on and in the thick terrace deposits forming the upper elevations of the right abutment. Landward (south) from about station 31+00, the right abutment is a nearly horizontal surface, which is the surface of an extensive alluvial terrace margining the San Gabriel River valley for approximately 13 miles upstream and 6 miles downstream from the dam. The right (south) side slope of the San Gabriel River valley descends from an elevation of approximately 540 feet at station 30+50 to the margin of the flood plain at about elevation 475 feet at station 40+50. The San Gabriel River meanders in its flood plain from beyond the upper end of Granger Lake downstream to beyond the end of the outlet and spillway discharge channels. The flood plain of the river is approximately 3,000 to 3,500 feet wide and is oriented nearly northeast through the lake, but begins to curve toward the east at about the location of the dam. Willis Creek, the largest

tributary of the San Gabriel in this area, leaves its own valley and enters the San Gabriel flood plain about 3,500 feet upstream from the axis of the dam. It joined the San Gabriel River 5,400 feet downstream from the axis of the dam (prior to backfilling its channel at and below the dam). The dam crosses a low level terrace between approximate station 84+00, near the left (north) bank of Willis Creek, and station 142+90 on the left abutment. This terrace is erosionally dissected by Sorefinger Creek and its unnamed tributaries. The outlet works is excavated through this terrace. Sorefinger Creek crossed the centerline of the dam north (left) of the outlet works at embankment station 108+60 and joined Willis Creek 2,900 feet downstream. The gentle slope down the left abutment continues south past the left extremity of the low level terrace to end on the terrace surface. The location at which the gentle slope of the left abutment joins the terrace surface between stations 133+00 and 135+00 is somewhat difficult to discern because of dissection of the abutment slope by the unnamed tributary of Sorefinger Creek.

b. Overburden. Bedrock in the upper slopes of the right abutment is mantled with 14 to 25 feet of terrace alluvium of Pleistocene age. This material is part of an extensive remnant of the San Gabriel River flood plain, or its predecessor, deposited much earlier than the present erosional stage of the river. The terrace in which the outlet works is located is a remnant of a stage of the San Gabriel River intermediate in age and in elevation between the terrace alluvium of the right abutment and the present flood plain. The intermediate terrace deposits have an aggregate thickness of from 40 feet to less than 10 feet near Sorefinger Creek and its unnamed tributary. The greatest thickness is in the vicinity of station 142+50, where the overburden is a little more than 40 feet thick. North from station 142+50 (toward the left abutment) the thickness of the overburden diminishes rapidly from 40 feet to approximately 10 feet in a horizontal distance of 50 feet. This was the site of the left cut bank of the San Gabriel River at its intermediate stage and marks the northern edge of the terrace alluvium. Overburden in the present flood plain varies in thickness from about 10 to 28 feet. Typically, the lithologic sequence comprising both the flood plain and the terraces is as follows.

(1) Clay (surface material)

(2) Clay (caliche - right abutment terrace only)

(3) Sand (local deposits)

(4) Gravel (locally interbedded with sand or clay)

(5) Shale bedrock

A local variant from the typical lithologic sequence described above occurs between station 58+00 and the backfilled San Gabriel River chan-

nel. Along this reach of the embankment foundation the overburden is composed of 15 to 20 feet of recently deposited sandy clay in the flood plain, occupying the position of (2) above. Shaly clay is another minor constituent of the overburden. Its appearance is that of highly weathered shale, but it lacks the structure of shale and has the consistency of clay. This is a transported material rather than one which is in situ. In three places where it is mapped, it is only distinguishable from underlying weathered shale by the presence of a few gravel between it and the shale. In the remaining four places in the cutoff trench where it was found, it is separated from the shale beneath by alluvium of somewhat greater thickness.

Overburden on the left (north) abutment between station 143+00 and the north end of the inspection trench diminishes from 10 feet to less than 5 feet in thickness. Here the overburden consists (from bottom to top) of a thin bed of residual clay, formed by extreme weathering of the shale bedrock, which in turn is overlain by very slightly gravelly alluvial clay. The slightly gravelly clay appears to have been deposited either as slope wash or as filling of minor rills.

c. Bedrock and Stratigraphy. Three bedrock lithologic units are recognized in the area of the dam which are designated, for purposes of convenience, as Shale I, Shale II, and Shale III. All three of these units are stratigraphically in the middle of the Taylor Group. Their description and correlant formations are as follows:

Shale I: (Uppermost) Dark gray where unweathered, calcareous with scattered more calcareous beds, soft to moderately hard. Correlated with the basal portion of the Marlbrook Formation ("upper Taylor marl").

Shale II: Gray where unweathered, very calcareous, brittle, with some less calcareous softer beds, generally moderately hard, average thickness 58 feet. Correlated with the Pecan Gap Formation.

Shale III: Gray to dark gray where unweathered, very sandy to sandy in the top 10 to 12 feet with sand content diminishing rapidly below, calcareous, with scattered more calcareous beds, moderately hard to soft. Sandy top 10 to 12 feet correlated with basal part of the Pecan Gap Formation or possibly with the Wolfe City Formation (sandstone). Shale strata below top 10 to 12 feet are correlated with the Ozan Formation ("lower Taylor marl").

Shale I comprises the bedrock of both abutments, but has been eroded from the flood plain and intermediate terrace between station 40+10 and station 123+90. Shale II comprises the bedrock between these stations and is also the foundation for the outlet works, excepting the

stilling basin and lower chute slopes which are founded on Shale III. Shale II is also the foundation for a small portion of the keys and wall footings of the left (north) side of the stilling basin of the spillway. The maximum thickness of Shale I in the spillway, prior to excavation, was about 80 feet. Its thickness after excavation is 50 to 60 feet at the location of the spillway refill section. The thickness of Shale I is probably between 125 feet and 135 feet at the north end of the dam (based on borings 8A6C-84 and 8A6C-85, located 3,000 and 1,500 feet from the north (left) end of the dam). Shale II varies in thickness from about 20 feet near Willis Creek to its full thickness of 58 feet. Shale III is more than 300 feet thick (based on two oil tests drilled immediately downstream from the dam in 1960).

Lithology of the basal few feet of Shale I is transitional with the underlying Shale II, principally in lime content. Lithology of the base of Shale II is also transitional into Shale III, but the transition is limited to a single bed which varies in thickness from 1.5 to 3.0 feet. This bed has the high lime content of Shale II and the sand content of the upper 10 feet of Shale III. Core description included this bed in Shale III, but it was more practical to include it in Shale II for mapping purposes during construction. Shale II contains a thin, micaceous, and generally soft clay shale bed having a relatively high content of montmorillonite (smectite). This bed is referred to as the lower depositional seam. The importance of this bed is out of proportion to its thickness in that its shear strength is sufficiently low to have affected the design of the embankment and the foundation of the outlet works. Generally, the shear strength of this depositional seam is lowest near water filled fractures. The lower depositional seam, varies in thickness from 0.2 foot to a maximum of about 0.9 foot, and is absent, by reason of erosion, in only a few places. It occurs approximately 23 feet above the base and 35 feet below the top of Shale II. Removal of the lower depositional seam from the foundation of the outlet works was required because of its insufficient shear strength. An upper depositional seam was identified and later monitored for pore pressure by 5 piezometers, completed under the first contract. This unit is approximately 30 feet higher in the bedrock sequence than is the lower depositional seam. Consequently it is very near the top of Shale II. No additional piezometers were installed in the upper depositional seam under subsequent contracts as the unit is missing in much of the bedrock of the valley section because of erosion.

Petroleum is a minor constituent of both Shale I and Shale III. A number of oil-spotted horizons parallel to bedding and a few petroliferous seams are present in Shale I in the spillway, principally in the lower slopes of the chute. Typically these appear as either lines of brown or tan dots of staining or as thin lines of staining. In addition to being oil stained, Shale I bled crude oil from a gently dipping parting in the bottom and another in the backslope of the cross drain at spillway station 32+01.3 between offsets 406.2 feet right and 419.9 feet right. A similar parting bled crude oil from the backslope of the same

cross drain between offset 447.7 feet right and the key cut at the right (south) spillway wall footing (o/s 475 feet right). Core from the upper 10 to 12 feet of Shale III bled natural gas from blebs and streaks of soft, oil stained sandstone within the shale. A strong petroleum odor permeated the stilling basin of the outlet works as this same portion of Shale III was excavated. No hazard to man or machines developed from this, and the fumes dissipated shortly after cessation of excavation.

d. Bedrock Structure. In addition to southeastward regional dip, three types of bedrock structure, namely flexure, faulting, and jointing, are present under the dam and in the vicinity. The dam crosses a structural arch, or possibly an anticline, the areal configuration of which has not been defined. The geologic section along the centerline of the dam (plates 6, 7, 8, and 34) indicates the apex (point of change from south to north dip) of the arch-like structure to be near the outlet works. The flexure has at least 50 feet of vertical relief between the spillway and the left (north) abutment, which is believed to exceed that which might be expected from elastic rebound because of the San Gabriel valley. This flexure is not a simple one. Rather, it is considerably modified by faulting.

Two principal fault systems are recognized at the dam, each having its own characteristic strike, direction of dip, and typical magnitude of displacement. One fault system, which appears to be part of the regional Balcones system, dips downstream. Its faults strike from 10° to 18° northeast of the alignment of the dam (a strike of north 32.7° east to north 40.7° east). Faults of the second system strike sharply across the centerline of the dam. Faults with Balcones orientation generally have greater vertical displacement than do those of the second or local system, displacements usually varying from 10 to 50 feet. Subsurface mapping and geologic cross sections at the damsite suggest that faults of both the Balcones and the local systems do not extend long distances without losing displacement or dividing into two or more faults.

Only two faults of Balcones orientation and displacement are believed to cross the centerline of the dam. One of these crosses the outlet discharge channel less than 100 feet downstream from the end of the stilling basin. See figures 36, 37, 38, 39, and 42, also plate 30. Here its displacement is approximately 40 feet. It appears to extend southward, crossing the centerline of the cutoff trench at station 69+10, where it has a probable displacement of 6 feet. This fault which dips downstream, is present along the shale bottom of the cutoff trench for a distance of 345 feet because of the close relationship between the azimuth of the trench and the strike of the fault. See figures 10 and 11, and plate 14. Another fault, probably a splinter of this fault, crosses the centerline of the cutoff trench at about station 70+90, and appears to join it beneath the overburden comprising the right sideslope of the trench. Displacement of the splinter fault is nearly the same as that of the parent fault (Balcones) based on geophysical logs of nearby

borings. The second fault with Balcones strike, and downstream direction of dip, believed to cross the centerline of the dam, has a displacement from 25 to 37 feet, depending upon location. It was discovered and traced solely by geophysical logs of borings in the right abutment. This fault could not be located in the trench due to the highly weathered nature of Shale I in this portion of the trench. It is shown as a dashed line on the map of the cutoff trench, plate 12, at its probable location at station 30+00.

Five minor faults are present in the right (south) sideslope of the outlet stilling basin, having strikes very nearly the same as those of the Balcones system, but having an opposite direction of dip (upstream). See figures 36, 37, 38, and 39, and plate 30. These faults have maximum observed displacements of 2.0 feet (at the lower depositional seam). Their displacements diminish downward to disappearance of the faults before the Shale II/Shale III boundary is reached low in this sideslope. These faults, while having strikes much the same as those of faults of the Balcones system, may have developed earlier than those of the Balcones system, as one of them was observed to be terminated by the Balcones fault just downstream from the outlet stilling basin (figure 38). As far as can be discerned, these faults are present only in Shale II. Shale II is more brittle and susceptible to faulting than is Shale III below or Shale I, which once overlaid Shale II at this location.

The second system of faulting recognized at the dam is distinguished from the system having Balcones orientation by the strike of its faults, their upstream dip, and their generally smaller displacements. Faults of this system appear to be related to the arching of the bedrock strata at the dam. For purposes of convenience, they are considered to belong to a local system. Influence of the San Gabriel valley flexure on this system is indicated by the fact that the vast majority of these faults dip toward the apex of the arch which is near the outlet works. Faults located south of the apex dip north toward the apex, and faults north of the apex dip south toward the apex. Faults of the local system cross the alignment of the dam at angles of from 60 to 85 degrees from the centerline north of P.T. station 30+89.76. These angular crossings of the centerline by faults of the local system also suggest a relationship between the local fault system and structural arching of the bedrock strata. Most of these faults south of the apex of the arch are so oriented that in the upstream direction their 60° to 85° angular relationship with the centerline is toward the south (right) abutment. North of the structural apex, this angular relationship is usually reversed (toward the north abutment). The preponderance of faults of the local system have vertical displacements from 2 to 11 feet (based on borings and on seam offset in the spillway). However, three of them have displacements of as much as 17 to 22 feet. Most of the local faults have relatively consistent or straightline strike, so far as can be seen in excavations or interpreted from borings. Nonetheless, there are exceptions. Twelve faults of the local system were found crossing the cutoff trench. An additional 10 probable and possible faults were

recognized, either in the cutoff trench or by means of borings. Three faults of the local system were mapped in the spillway excavation (see plates 32 and 33).

A series of striated and slickensided fractures were found intersecting faults in Shale II, which may be described as scallop-shaped, cuspate, or conchoidal. Displacement, which produced the striations, was seldom discernable, but in those instances where it was, it usually was less than 0.1 foot, and never more than a few tenths of a foot. Fractures of this type were found associated only with faults and not to cross to the opposite side of the fault they intersected. Their usual orientation is from about 30 degrees to 45 degrees from the strike of the fault. This correlative arrangement was sufficiently consistent that the finding of one of these fractures generally brought about recognition of a fault. The angular relationship of these fractures to their associated faults suggests that fault displacement in this area may have a strike-slip component of movement in addition to its normal dip-slip movement. The dimensions of these fractures varies somewhat, but is usually from a few feet to more than 10 feet in some instances. Excavation through these fractures usually resulted in creation of slide blocks if the fractures were in a slope. In quite a number of instances additional fractures of this type were present inside and intersecting the larger scallop-shaped fractures. The effect of this locally was a shattered shale matrix (much of what was described in boring logs as brecciation is thought to be a shattered shale of this type). The largest number of and the best exposures of these fractures was in the outlet works excavation, principally because the outlet works was the most extensive and deepest excavation within Shale II. See figures 19, 20, 49, and 50. Fractures of this type, if present, were poorly developed in both Shale I and Shale III. Shale II, being the most brittle of the three shales at the dam, is more likely to have failed by fracturing than by distortion or dilation.

A considerable number of partings and a few very thin seams of soft shale were found in unweathered Shale I in the spillway. Easiest detection of both was on the freshly excavated and airblown foundation surface for the slab on chute slope and on the bottom of the stilling basin. See figures 69, 76, and 77. The partings are fractures developed because of a tendency of the shale to part in planar surfaces. This tendency appears to be related to past deformational stresses causing the clay particles in the shale to be reoriented in one principal direction. It was not certain whether these fractures were created by unloading or by the process of excavation, but the distribution and orientation of partings in the sideslopes of the spillway end sill key very strongly suggest that at least a large portion of the partings existed prior to excavation. Here, as in the bottom of the stilling basin, most of the partings appeared to be related to fault structure. While partings were fairly abundant in the spillway chute slope, they were most obvious where dessication had been allowed to occur between rough grading and fine grading (this phenomena is

discussed further under the caption EXCAVATION PROCEDURES). Partings were never observed to be slickensided, indicating tensional fracturing rather than shearing. Partings and seams on the spillway chute slope could be followed to their termination in undisturbed shale in most instances. The spillway chute slope provided a fairly good sample of the vertical distribution of the partings in unweathered Shale I. The bottom of the stilling basin, together with its keys and drains, gave a reasonably good sample of the horizontal distribution of partings. Dips of the partings in the chute slope that extend the greatest distances range from about 3° to 7°, although a few dips were slightly more than 10°. Dips of partings extending shorter distances tend to range from 15 degrees to as much as 27 degrees. The vertical distribution of partings in the chute slope did not display a consistent pattern. Pattern consistency was better on the bottom of the stilling basin where the dip of the partings generally ranged from 5 to 10 degrees. Parting dips were generally greater than the dip of the bedding everywhere bedding could be observed. This was especially noticeable near the downthrown side of faults. Near the fault closest to the right (south) side of the spillway, for instance, the dip of the partings increased to about 30 degrees at the fault, but the dip of the bedding was a little over 10°.

Seams of soft Shale I noted in the spillway are extremely thin (0.005± foot) and may consist of minor softening of the shale along a bedding surface. There are only a few of these present, their recognition generally being restricted to the chute slope of the spillway. The two most notable of these, separated 3.9 feet vertically, extended from the left sideslope beyond the left wall, inward across the spillway to within 210 feet from the centerline of the chute. Their presence caused benches to be formed in the slopes by horizontal wedge-shaped pullout of shale blocks during excavation. See figure 66 and plate 33. These seams, where offset by the farthest left (north) fault, made accurate measurement of the vertical displacement of the fault possible (11.3 feet).

A number of fractures with a maximum variance of dip of from 9° to 20° to the south were found in Shale I on the bottom of the cutoff trench in the right (south) abutment. Though mostly iron-stained or iron-encrusted, they are in an unweathered shale matrix between stations 36+31 and 39+88. A series of thin gypsum seams were found in the weathered shale in the bottom of the cutoff trench between stations 31+24 and 35+56. These seams have the same dip as the gently dipping fractures and have the appearance of fractures which had been widened by gypsum crystal growth. It appears likely that the gently dipping fractures and gypsum seams represent shears developed from relief of lateral confinement by erosional excavation of the present stage of the San Gabriel valley. It is also possible that the fractures and seams resulted from shears under the ancestral flood plain, which is now the abutment terrace.

Joints, meaning essentially vertical tension fractures, which are at nearly 90 degrees to approximately horizontal strata, are a relatively

common bedrock structural feature at the dam. Generally, they are not distributed in patterns considered significant to the foundation. Joints were mapped only where there was a significant number of them, usually in some pattern. There are four locations in the cutoff trench where they are shown: Between stations 42+58 and 43+36, between stations 85+00 and 86+00, between stations 106+77 and 107+50, and between stations 122+18 and 123+30.

A large number of steep-dipping, rust-filled fractures cross the cutoff trench between stations 108+17 and 109+35. A few possible faults, based on limited evidence, are also mapped in this portion of the cutoff trench.

e. Bedrock Weathering. Weathering of bedrock, in a technical sense, means degenerative alteration of its properties such as hardness, color, and internal structure due to action of the atmosphere or percolating rain. Mapping of the cutoff trench and the structural features of the dam required the following somewhat simpler definition. The foundation shale of the cutoff trench and outlet works was classified as unweathered if its matrix was essentially unweathered and it incorporated no weathered layers or zones. Under this classification, weathered (color-altered) margins of joints, faults, and other fractures could be included in shale mapped as unweathered, so long as they did not constitute the greater portion of the rock. Two degrees of bedrock weathering (weathered and slightly weathered) were mapped in the spillway. The boundaries of shale having these degrees of weathering are shown on the geologic map of the spillway, plates 32 and 33, by lines and lettering, but all weathering is lumped together for coverage by a single map pattern symbol. Two degrees of weathering were differentiated in the spillway because of the following factors: Weathering reaches much greater depths in the shale of the spillway and is more complex than it is in the shale of the valley section; the spillway is the deepest and most extensive single excavation at the dam, fully exposing the weathered portion of the foundation shale; and complete removal of weathered shale for replacement by refill required a practical field definition of weathering. Degree of weathering of spillway shale was classified as follows:

(1) Weathered: Wholly weathered or, proportionately most of the interval (thickness) consists of weathered shale (as near the base of the weathered section).

(2) Slightly Weathered: Proportionately more of the interval consists of unweathered shale than of weathered shale. The base of this unit was usually at the bottom of the lowest discrete weathered zone or layer.

(3) Unweathered: Unweathered shale matrix. Usually faults, joints, and other fractures in the uppermost part of shale of this classification have marginal weathering, extending downward for a number

of feet. In a single instance, marginal weathering was found along a fracture deep in the unweathered shale in the rough-graded lower chute slope, near the centerline and just above the cross-drain at station 32+02. This evidence of deep weathering was subsequently removed during fine grading.

Thickness of the weathered shale in the right (south) abutment varies considerably, depending mainly on topography of the eroded top of the shale. Weathering tends to be deeper and more pronounced in the slope of the abutment probably due to unloading and increased ease of percolation of both rain and terrace ground water. Total thickness of the weathered interval of the shale in the right abutment varied from approximately 13 feet to as much as 49 feet (from exploratory borings). The thickness of weathered shale across the spillway at the backslope of the refill section (spillway station 28+60) was 21 to 26 feet on the left (north) side, and 14 to 23 feet on the right (south) side. Top of the unweathered shale at the base of this slope varied in elevation from 494 to 502. Top of the weathered shale (and bottom of gravel) is from about elevation 516 at the right end of the refill backslope rising to the north to riprap grade (elevation 517.25) near offset 100 feet right. Left (north) of this point the top of weathered shale rose to approximately elevation 520 in the left spillway sideslope before excavation. The contact of the weathered shale with the unweathered shale below slopes gently downstream, which is in the direction of descent of the original ground surface. The weathered shale under the flood plain and under the intermediate-level terrace, as far left (north) as embankment station 142+50, is thin, varying from none near the San Gabriel River to a little more than 2 feet, averaging 1 foot. Local exceptional thicknesses are in a few buried bedrock hills composed of weathered shale. Weathering in the shales exposed in the slopes of the outlet works excavation reaches depths varying from a few tenths of a foot to a little more than 2 feet, averaging just under 1 foot. Weathering along margins of fractures very seldom reached the bottom "flat" of the outlet works excavation. The thickness of the weathered portion of the foundation shale increases abruptly northward (left) from station 142+50 to station 142+90, varying from a few tenths of a foot to nearly 26 feet. Between these stations the top of the shale rises from approximately elevation 470 to approximately elevation 495 in what is interpreted to be the left cutbank of the San Gabriel River during deposition of the intermediate level terrace described earlier under "Overburden." Data on the thickness of the weathered portion of the shale northward from station 143+00 to station 159+00 are sparse. Weathering here, as in the right abutment, reaches its greatest depths in the shale. Excavation north of station 142+70 was for the inspection trench only and information on weathering in the bedrock below the bottom of the trench is limited to that from boring 8A6C-85, located on the centerline at station 146+50. At this location, the uppermost 42.7 feet of Shale I is weathered. Presumably weathering does not reach materially greater depths than this in the shale between the boring and the left end of the dam.

f. Ground Water. Alluvial gravel and sand, overlying bedrock in the right (south) abutment, contains a large volume of fresh ground water. The volume of water derives from the great lateral extent of the stream terrace. The water-saturated section within the confines of the spillway, however, was only about 2 to 3 feet thick. The prime and subcontractor's offices and their concrete batch plant, under the last contract, were supplied with water from a single well producing from the terrace alluvium a few hundred feet south of the spillway. Ground water was encountered in the sideslopes of the cutoff trench between the spillway and station 30+10 but not in the slope from the right (south) abutment to the flood plain. Ground water in the flood plain alluvium and in the intermediate-level terrace is thin. Only insignificant ground water flows entered the cutoff trench between the right abutment and approximately station 53+00. Some minor flows were encountered from the basal gravel between station 53+00 and station 56+50. As expected, good flows of ground water were experienced in excavating the cutoff trench between station 58+10 and the north (left) bank of the San Gabriel River. Only minor local flows with even fewer moderate flows were experienced in the cutoff trench between the San Gabriel River and the left abutment. Minor local flows were present to the north (left) as far as station 142+50 in the lower slope of the left abutment where the terrace sand and gravel end. No ground water was encountered north of this station. Control of ground water flows are discussed later under DEWATERING AND FOUNDATION DRAINAGE.

3. Engineering Characteristics of Foundation and Refill Material. Engineering characteristics of the materials comprising the foundation for the embankment, outlet works, and the spillway and for refill material were determined by laboratory tests made on samples of these materials. It was concluded from stability analyses that embankment base width and slopes must be designed for shear strength of the depositional seams in Shale II, even though weathered Shale I was the predominant foundation material in the abutments. It was also concluded that neither of the depositional seams of Shale II could be allowed to remain in the foundation of the outlet works. The following values were selected for design:

a. Overburden.

Moisture Content 26%

Dry Density 100 pcf

Shear Strength Tests

Type Test	C tsf	Ø Degrees
Q (unconsolidated, undrained)	0.45	1.0
R (consolidated, undrained)	0.4	10.
S (consolidated, drained)	0.	31.

(1) Assumed shear strength of spillway gravel and sand refill:

$\phi = 32°$

C = 0 psf

(2) Criteria for spillway and outlet:

Earth embankment: 123 pcf (moist)

125 pcf (saturated)

b. Shale I, Weathered.

Shear strength (from pre-split and residual direct shear tests):

10 to 12 degrees

Assumptions: Pore pressure of 30 percent and 50 percent for stability analyses of abutments.

c. Shale I, Unweathered.

Values used for redesign of spillway (at elevations below granular refill).

Allowable bearing pressure:	10,000 psf
Shear Strength:	∅ = 21 degrees
	c = 300 psf

Criteria for spillway and outlet:

Shale foundation	= 134 pcf

d. Shale II, Weathered.

No suitable undisturbed samples obtainable. Remolded material from test pit No. 2 was tested. No excess pore pressure anticipated because of small weathered thickness and because of proximity to overburden sand and gravel for drainage.

e. Shale II, Unweathered.

Highest shear strength of the shales at the damsite. Did not control design. Unconfined compressive strength 9 to 38 tsf.

f. Shale II, Depositional Seams, Unweathered.

Shear strength of the depositional seams controlled the outer slopes and consequently the width of the dam. The embankment slopes were designed to be stable with 30 percent pore pressure in a depositional seam located 5 feet beneath the top of the shale. All depositional seam material was to be removed from beneath the outlet works. Samples of undisturbed seam material were unobtainable in sufficient size for testing. A realistic estimate of shear strength of seam material for design use was:

∅ = 13 to 15 degrees

c = 0

g. Shale III.

Though undisturbed Shale III was tested to some extent, strength values for design purposes were not reported in Supplement No. 3 of Design Memorandum No. 4. The principal reasons for this were: (1) Only one structure, the stilling basin of the outlet works, was founded on

Shale III; (2) being a generally sandy material, its ϕ angle was fairly large; and (3) shear strength of its much less sandy part was not materially different from that of the other two unweathered shales.

4. Unanticipated Geologic Conditions Encountered During Construction.

a. Cutoff Trench. On 9 July 1977 the cutoff trench was incompletely excavated from about station 134+00 to station 143+25 and was approximately 5 feet below grade at station 142+00 (approximately elevation 483) without having reached shale. The top of the shale sloped southward, disappearing in the bottom of the trench at approximately station 142+90. The contractor (Abrams) elected to dig two pits with his backhoe to find shale, one pit being located at station 141+94 and the other at station 139+50. Top of the slightly weathered shale was found at elevation 468.2 at station 141+94, and at elevation 468.1 at station 139+50, meaning that excavation of an additional 15 feet would be necessary to accomplish cutoff at the site of the first pit and 6 feet at the site of the second pit. Specifications required a change of slope of the trench sides from 1 vertical on 2 horizontal to 1 vertical on 3 horizontal due to the increased depth. The altered section of directed trench excavation was to commence at station 143+80 and terminate at approximately station 134+00. The result was an abrupt widening of the trench where the shale dropped precipitously southward (station 142+90) with subsequent narrowing southward from about station 142+00 as the ground surface descended toward the nearly horizontal top of shale, to transition into the unmodified, partially excavated trench at station 134+00. See plates 18 and 19 for this reach of the trench. The top of the shale, as found, differed from that shown in the plans because insufficient subsurface data were developed in this reach. Two borings controlling this part of the centerline profile (8A6C-84 and 8A6C-85) were drilled 1600 feet apart. This required that the profile be an interpretation of the configuration of the top of shale between the borings where the top of shale differs in elevation by 43.9 feet. In retrospect the drilling of a few supplementary auger borings would have been economically justified to reduce the risk of a claim.

A series of gypsum seams were found in the shale of the right abutment in the bottom of the cutoff trench (previously described). Since gypsum is subject to solutioning by ground water unsaturated in calcium sulfate, a decision was made to excavate the cutoff trench 3 feet below grade from about station 35+50 to station 30+00 in hope of eliminating the gypsum seams from the cutoff trench foundation. Excavation was to return gradually to planned grade between station 30+00 and station 25+00. This amount of additional excavation was insufficient to rid the foundation of all the seams.

b. Outlet Works. The outlet excavation was first inspected on 23 July 1974 at a time when only the downstream end of the extensive area of the "flat" had been excavated, and before the first conduit concrete had been placed. See plate 30 for the plan of this part of the outlet

excavation. The lower depositional seam of Shale II was found in the steep inner sideslope on the south (right) side of the excavation 2 feet or less above the "flat," as had been anticipated. However, it was also found in the sides of the "V" of the rough-cut conduit excavation in the vicinity of station 14+60, where it was not expected to be, and was missing in the shale of the steep north sideslope of the outlet works excavation where it should have been present. The seam was traced along the steep south sideslope as far upstream as station 13+93, where it disappeared at a steeply dipping fracture. In addition, there was also a large cuspate fracture at station 13+93, which caused a large block of Shale II to slide out of the slope. See figures 20 and 21. Presence of the seam in the "V" of the conduit indicated the necessity of overexcavating for its elimination from the foundation. Absence of the seam in the steep inner north slope of the excavation meant the seam was below the grade of the "flat" from the conduit to the north sideslope. Further, termination of the seam at a fracture in the south slope strongly suggested down-faulting upstream with the seam below grade across the whole width of the outlet "flat."

These conclusions prompted the digging of eight shallow backhoe pits in the shale "flat," located on both sides of and away from the conduit. The pits were from 4 to 6 feet deep and were intended to locate the lower depositional seam. In addition to recording the presence or absence of the lower depositional seam in the pits, a record was made of any fracturing found in the shale and of any water encountered during excavation or of any water subsequently entering the pits. Water seeped into three of the pits after their excavation and one additional pit was described as being wet. This investigation was not successful in determining the detailed configuration of the lower depositional seam, but did prove that its configuration as shown on the plans, was incorrect. Also, considerable fracturing was present in part of one pit, the pit being long and angularly oriented, with respect to the centerline, between station 13+97 and station 14+48 about 70 feet right of the conduit centerline. It was later recognized that the fault shown at station 14+48 (at the centerline) on plate 30 crossed the pit and was responsible for the fracturing. The configuration of the seam shown on the plans being in error, and removal of the seam being required, it became mandatory to re-explore the foundation. Consequently, a trench was dug with a tractor-mounted backhoe along the centerline of the conduit from the site of the headwall, upstream as far as the backhoe could reach th lower depositional seam (station 13+00). Trenching revealed the presence of two faults crossing the outlet alignment within a horizontal distance of 200 feet. The lower depositional seam was found to be within the elevation range of the "V" bottom of the conduit collars, thus below grade of the "flat" for the whole length of the trench. Re-exploration by pit and trench resulted in a delay in the intended start of conduit concrete placement of approximately one week.

On 1 August 1974 a decision was made to remove the depositional sea from the foundations of the conduit, gate tower, and approach slab, as

originally specified, but not to remove it from under the "flat" area surrounding these features. Also, it was decided that the remainder of the foundation for these features would be re-explored to locate the lower depositional seam. Re-exploration was accomplished by core drilling along the centerline of the conduit, and by drilling one boring on each side of the upstream end of the conduit, the tower, and the approach "U"-wall structure. The unexpected configuration of the lower depositional seam, and the necessity of its removal, required changes both of grade along the conduit alignment and of profile across the conduit excavation. Explorations conducted during construction showed the lower depositional seam to be at or below grade of the bottom of the conduit protective concrete between station 7+70 and a fault at station 8+73, between station 12+70 and a fault at station 13+30, and between station 13+80 and a fault at station 14+48. See figures 22, 23, and 24. The depositional seam was within the elevation range of the "V" bottom of the conduit, shown on the plans, throughout the remaining length of the conduit, except between about station 9+18 (left side) and about station 10+20 (right side) where the seam is above the top of protective concrete for the "V" and consequently above the elevation of the "flat."

The grade of the bottom of the conduit excavation, as redesigned, departed from the original grade at station 7+35 with a descending slope two monoliths long (25 feet each) to station 7+85. From station 7+85 downstream to station 9+10 the new grade remained horizontal at elevation 449.0. It rose uniformly to original design grade between station 9+10 and station 9+35 with the original grade of the plans retained between station 9+35 and station 12+10. From station 12+10 the redesigned grade descended gently from the original grade to elevation 447.0 at station 12+60. The new grade remained horizontal from station 12+60 to station 13+85, where it began another descent to station 14+35 (the length of two monoliths). It remained horizontal at elevation 445.0 from station 14+35 to station 14+60 from which point it ascended to elevation 446.3 at station 14+85. The grade remained horizontal from station 14+85 to the headwall excavation at station 15+05. Changes of grade and profile across the conduit excavation were made at the collars to obtain uniform change through each monolith. The lower depositional seam was found to be above grade of the plans in the area of the gate tower, thus no change of grade was required. An angularly oriented fault was found to cross the foundation of the approach slab and walls. However, in this case it was decided to use the grade of the plans, leaving the lower depositional seam beneath the upstream portion of the approach structure. This was justified because loading of the upstream portion of the approach foundation by the "U"-wall structure was relatively light (low wall height). Where the redesigned grade of the conduit bottom was deeper than the planned bottom of the "V", the bottom of the excavation was cut horizontal across the full width of the conduit. Sideslopes here were designed for a slope of 2 feet vertical to 1 foot horizontal, but were frequently cut steeper. Where the grade of the bottom returned to that of the plans, but where the seam was still within the depth of the protective concrete under the "V", excavation

was full width of the conduit to a depth sufficient to remove the seam, that depth being the depth to the seam on the lowest side. Below this depth the bottom was to be sloped inward to meet the original grade of the protective concrete under the truncated "V". Where the grade on the bottom was to be that of the plans, but where the depositional seam was above the top of the protective concrete of the "V" (and the grade of the "flat"), the excavated profile across the conduit was as shown on the plans. Except for those portions of the conduit where the plans were to be followed unmodified, the protective concrete was changed to backfill concrete, the top surface of which was screeded to the configuration of the structural concrete to follow. Backfill concrete was specified to give an unconfined compressive strength of 2000 pounds in 28 days time. Where full width excavation to remove the depositional seam was necessary, width of the excavation was a minimum of 26 feet for the conduit monoliths and 32 feet for the collars.

The principal reason why grades for the outlet works required modification was insufficient allowance for possible variations of nature. Design of the foundation of the outlet works assumed the base of the lower depositional seam to be at elevations higher than those of the foundation. Thus the seam was expected to be removed during excavation. However, of the three exploratory borings along the centerline of the outlet works, boring 8A6C-593, at station 5+10 at the approach structure, had encountered the base of the depositional seam 0.6 foot below the grade of the top of the "V" of the conduit as extended. Boring 8A6C-594, at outlet station 10+00, encountered the base of the seam exactly at grade of the top of the "V" of the conduit, and boring 8A6C-595, at station 16+60 in the stilling basin, encountered the base of the seam 1.5 feet below grade of the top of the "V" of the conduit, as extended. As subsequently discovered, each of these three borings were located on the high part of a tilted fault block. If at least one of the borings had been on the downthrown side of one of the faults, additional exploration for the depositional seam would probably have been done, and at least the conduit and possibly the "flat" design would have been adapted to geologic conditions before plans were issued. The fact that all three borings were located on the high portion of separate fault blocks, rather than being located at various positions on the unseen fault blocks, was purely accidental. To be reasonably certain of recognizing geologic phenomena of this kind, the distance between borings should be smaller than the dimensions of the feature to be detected. Otherwise, in areas where topography and outcrops do not reveal geologic structures present, only the laws of probability will determine whether features will be recognized and delineated.

EXCAVATION PROCEDURES

1. Cutoff Trench. No unusual construction methods were required to excavate the cutoff trench. Typical equipment used was scrapers, push cats (dozer with special blade), bulldozers, and blades (road maintainers). J.D. Abrams Company used a large caterpillar backhoe to top load wet, fine sand (into scrapers) from the bottom of a short length of the trench north of Sorefinger Creek, and for cleanup in the closure section between station 60+15 and station 61+40. Dahlstrom Company used a farm tractor-mounted backhoe to excavate a "notch" in the shale of the trench bottom along the fault located at station 49+46. This was to remove some of the intrically fractured (brecciated) shale marginal to the fault. They used a Northwest 80-D dragline to some extent for trench excavation between station 40+00 and station 57+50, and used it extensively for trench excavation between station 57+50 and station 60+45. In addition to bulk excavation, much of the trench sideslopes between station 57+50 and station 60+45 were excavated with the dragline, particularly along the right side. Initially, Dahlstrom excavated the cutoff trench between station 41+00 and the dike protecting the trench from the river immediately north of station 60+45. The trench was cleaned up and backfilled working northward from station 41+00 toward the San Gabriel River. The inspection trench from station 1+90 to the spillway was excavated and backfilled concurrently with the valley section of the cutoff trench. The cutoff trench from station 41+00 to the spillway was excavated during the latter stages of backfilling the valley section of cutoff trench and during the early stages of construction of the impervious section of the embankment. J.D. Abrams commenced cutoff trench excavation at the south side of the backfilled excavation of the outlet works, working southward to Willis Creek. Shortly thereafter, the cutoff trench was excavated from adjacent to the San Gabriel River, northward to Willis Creek. Next, the cutoff trench was excavated from the north side of the backfilled outlet works excavation northward to Sorefinger Creek at station 108+60. After a period of about 4 months had elapsed, cutoff trench construction was resumed from station 117+00 southward to Sorefinger Creek. During this period, much of the upstream random and semi-compacted zones of the embankment were placed. At the end of this period these upstream zones extended continuously from the outlet works, northward across backfilled Sorefinger Creek to meet an upstream section of embankment built under the previous contract (outlet works). Sorefinger Creek had become a lake and had risen, overtopping the upstream embankment fill. See figures 13, 14, and 15. Both the deeper portions of the clay fill upstream and the clay overburden beneath it were discovered to have been water softened, apparently from water in the underlying gravel under a head from lake Sorefinger. Consequently, additional excavation of rectangular shape was done upstream to remove much of this material (see plate 17 for the plan view of this excavation). When clay backfill in the trench at station 108+05 was removed, along with the adjacent softened clay, a "crack" was reported to have made bubbles and a small amount of water between the downstream toe and the centerline of the

trench (no water or bubbles detected upstream from the centerline). The next section of trench completed was from station 143+25 to the north end of the inspection trench followed by construction between station 143+25 and station 117+73, including the section requiring overexcavation to accomplish cutoff described earlier. The final stage of construction was that of the closure section between station 64+50 and station 60+15.

Trench construction between station 110+00 and station 111+00 involved a minor amount of overexcavation to accomplish cutoff in shale. Initial excavation exposing the shale disclosed several highly weathered bedding partings which caused slabs of shale to break out. The surfaces of the partings were wet, indicating seepage, and were eroded sufficiently that the surfaces did not match when pressed together. The partings were in the south-sloping shale just north of Sorefinger Creek. The shale slope was overexcavated sufficiently northward into the slope that the bedding partings were dry and mated perfectly when pressed together. Since the partings would mate perfectly, it was concluded that significant seepage should be precluded after application of the load of trench backfill and the embankment above it.

Backfill of the outlet works, placed under the second contract (Hensel Phelps), had been modified to the extent of leaving low, incompletely backfilled areas on the cutoff trench alignment on each side of the conduit, intending to save re-excavation and replacement later. The plan of these areas is shown as a detail on plate 5. The effect of this was to create rain collecting ponds which existed for several months. The pond on the north side of the conduit had existed for sufficient time for the clay beneath it to become quite moist and soft. This material was removed down to the shale "flat" of the outlet excavation, and replaced with clay of proper moisture content. This was done when the cutoff trench was excavated north from the outlet works.

2. Outlet Works. Equipment appropriate for common excavation was employed in construction of the outlet works, i.e. scrapers, push cats (dozers with special blades), bulldozers, and blades (road maintainers). This applies to both the contract for construction of the outlet works and to the final contract involving the approach and discharge channels. The excavation for the outlet works was, in essence, a flat-bottomed area 330 feet wide, 1,505 feet long, and 40 feet deep, into the bottom of which the approach structure, gate tower, conduit, chute, and stilling basin foundations were excavated. The sideslopes were designed to be 1 vertical on 3 horizontal through the overburden and 2 vertical on 1 horizontal from elevation 463 down to the shale "flat." Part of rough excavation for the conduit, as modified, was done with scrapers. Fine grading for the concrete structures was done with a T-725A Bantam gradall. Initial excavation of the broad "flat" area was concentrated in the downstream half of the outlet works in order to allow earliest placement of concrete (downstream conduit monoliths). From there, excavation of the "flat" proceeded upstream, ahead of the need for conduit

grading. The sideslopes and rough-graded bottom of the chute and stilling basin were excavated during the early stages of completion of the conduit foundation. The gate tower foundation was also excavated before completion of the conduit foundation. The conduit foundation was completed monolith by monolith, working upstream, leaving gaps for the later excavation of collars. Finish grading of the chute and stilling basin proceeded downstream starting at the headwall of the conduit. The contractor (Hensel Phelps) used a work slab covering the sand of the drainage blanket of the chute and stilling basin from which to drill anchors and on which to place chairs to support slab reinforcement. (The foundation excavation had been deepened by the thickness of the work slab). The cost of the work slab was borne by the contractor.

3. Spillway. Scrapers, push cats, bulldozers, and blades were used to excavate the spillway, both under the first and last contracts. A caterpillar backhoe, Model 235, was used to fine grade the chute and stilling basin slab foundations. This piece of equipment was provided with a "knuckle" for angle cutting so as to diversify its excavating ability. A smooth-mouth blade was used on the backhoe, in place of bucket teeth, for finish grading. The wall footings were fine graded with a gradall. Keys and parts of the cross-drains were excavated largely with a gradall, only minor lengths of these being excavated with a small backhoe.

The first excavation of the spillway foundation beneath the concrete structure was for the left (north) end of the refill section, extending from about 250 feet left (north) of the spillway centerline to the left sideslope. Excavation of the refill section was progressively extended to the right (south) from the initial area of work. Rough grading of the left wall footings and a small adjoining part of the chute slab was done immediately after the adjacent part of the refill excavation upstream was completed in order that construction of the left wall might commence as early as possible (rough grading meant leaving a 1 foot thickness of shale above finish grade). Rough grading for the chute slab was done working from the left wall footings toward the centerline for an initial distance of 100 to 150 feet. One foot of shale remaining above finish grade proved to be insufficient protection from desiccation of the foundation for a period of 3 months before finish grading. Finish grading inward from the left wall resulted in an abnormal amount of pullouts of shale blocks with a consequent high degree of roughness of the excavated surface. As a result of this experience rough grading procedure was changed to leave no less than 2 feet of unexcavated shale above finish grade. The new procedure, plus a shorter delay period between rough and fine grading, materially diminished shale desiccation, consequently reducing roughness of the finished shale surface. The left wall footings were progressively fine graded down the chute slope, reaching nearly to the downstream end of the walls before commencement of chute slab fine grading. Fine grading of the chute slab foundation was done in strips about 20 feet wide, but occasionally 40 feet wide, oriented down the chute slope. Grading commenced at the left wall,

progressing south to the right wall. Rough grading of the right (south) wall footings was done on completion of the refill excavation. Fine grading of the right wall footings was initiated while fine grading of the chute slab foundation was still in progress. Construction of the chute structural slab was well advanced before fine grading of the stilling basin foundation was commenced. Here again, grading began at the left side working toward the right side of the structure. Contract plans did not call for construction of a work slab covering the drainage blanket, through which anchor drilling could be done without disturbing or contaminating the drainage blanket sand, and on which support for slab reinforcement (chairs) and concrete forms could be placed. Because of the size of the spillway and consequent cost, the concrete subcontractor (Clearwater Constructors) chose not to provide a work slab at his cost, distinct from the structural slab, as he had done in constructing the chute and stilling basin of the outlet works. Therefore, at his request, the contracting officer approved placing the bottom 4 inches of the structural slab as a work slab. Later it was to be cleaned and the remainder of the required structural concrete bonded to it.

Initially, the cross drains in the chute were excavated after the drainage blanket had been placed. See figures 59, 60, and 61. Excavating the cross drains as a part of fine grading the chute slope was found to be more efficient, so separate excavation of drains was discontinued. See figures 62, 63, and 64. In contrast, none of the cross drains in the stilling basin were excavated as a part of finish grading the slab foundation. They were excavated after filter blanket sand had been placed, but before the work slab was constructed. The procedure for constructing keys in the chute was to excavate and place reinforcement and concrete through a breakout lane in the work slab. It was observed during inspection of keys, both in the chute slope and in the stilling basin, that the top 1 foot of the key sideslopes just below the drainage blanket was nearly always significantly rougher and more broken than was the remainder of the key sideslope below. This was observed both where desiccation of foundation shale was a problem and where it was not. The impression produced was that either the use of scraping tools to excavate the chute and stilling basin foundation caused a series of discontinuous breaks throughout the shale immediately below the scraped surface, or that the breakage observed was confined to the local effect of key excavation. If the former interpretation is correct, then designs for shale foundations should allow for somewhat diminished shear strength in approximately the first foot of the foundation.

DEWATERING AND FOUNDATION DRAINAGE

1. Cutoff Trench. Dahlstrom Corporation controlled ground water emissions from the sides of the cutoff trench solely by plating them with clay, and in so doing increased the difficulty of mapping. See figure 5. Plating was adequate to prevent entry of ground water where employed between the right abutment and about station 58+10, north of which the sand and gravel in the sideslopes were charged with river water. Here, foundation approval and mapping could be done only after pumping, and/or by pushing a wall of soil along the trench to displace water, together with plating the worst water flows with clay. Necessarily this work was done in short lengths of trench. The only other section of the cutoff trench south (right) of the San Gabriel River requiring control of ground water emissions was on the top of the right abutment between station 28+00 and station 30+10. At that location plating was only marginally effective. Under the Abrams contract (final) ground water flow was controlled by ditching the upstream toe of the trench, and by draining each section of trench toward the natural surface streams crossing the trench. Backfilling of the cutoff trench was conducted in the same manner, working toward the natural drainages, backfilling the stream channels last. Each surface stream channel crossing the trench alignment was backfilled upstream through the embankment area prior to excavating the cutoff trench, but was left unfilled downstream to allow drainage. Rarely did Abrams have to sump and pump. When required, it was usually in a local depression on the top of the shale, or as a part of closing a stream crossing. Trench backfilling procedure consisted of keeping the upstream drainage ditch open until clay backfill on the dry bottom of the trench exceeded the height of the water-bearing gravel along the upstream slope of the trench. The ditch was then cleaned and filled with compacted clay, and backfilling of the trench resumed.

Dewatering the San Gabriel River channel was an integral part of backfilling the closure section. Backfilling commenced upstream at the river diversion and proceeded downstream. The method typically utilized two bulldozers pushing a wall of soil down the channel to clean the bottom and sides. Movement of the wall of soil was immediately followed by placement and compaction of backfill clay by two other bulldozers. One of the defects in this procedure was that the cleaned bottom and sides of the channel were never visible during the operation. The cutoff trench was then excavated through the channel backfill. Old tires, a large amount of gravel, and poorly compacted clay were exposed in the sides of the cutoff trench and in test pits where the sides of the San Gabriel River channel met the channel bottom. See figures 16, 17, and 18. Scrapers running over portions of the channel backfill downstream from the cutoff trench caused the fill to "pump" or appear as if the scrapers were running over a water bed. Those areas where the fill had "pumped" and areas around test pits where debris was found, principally along the sides of the channel, were re-excavated to remove the debris and replace it with suitable material.

2. Outlet Works. The "flat" bottom of the outlet works excavation was maintained in a dewatered condition by means of two peripheral ditches, one on each side (left and right). Each ditch commenced at the access road entering the upstream end of the excavation and terminated at a sump at the downstream end of the "flat." Each sump consisted of a shallow excavation in the shale, surrounded by a low dike on three sides. The dike around the right sump was composed largely of compacted shale debris rather than of clay. Such composition resulted in continuous slow leakage down the right side of the adjacent chute slope. Water levels in the sumps were limited by a float-controlled electric pump which discharged outside the outlet excavation. No interceptor ditches were necessary at the contact between the gravel and shale in sideslopes of the chute/stilling basin excavation, but sumps were dug on both sides of the excavation beyond the ends of the walls to hold rain runoff. Though no ground water was encountered in the shale during excavation of the chute and stilling basin, the foundation is drained by a filter sand blanket 12 inches thick, extending downstream as far as station 17+13. The drainage blanket discharges into two cross drains and into gravel collector drains along the outside edges of the structural concrete. The collector drains were constructed concurrently with the drainage blanket, consequently requiring protection from contamination and disturbance during the subsequent period of concrete construction. The contractor (Hensel Phelps) accomplished this by placing a thin concrete cover slab over the full length of the external collector drains. The value of the slab was soon evident. More than 4 feet of silt and mud collected outside the walls of the chute and stilling basin from runoff during the remaining concrete construction. The drains, after clean up and removal of their cover slabs preparatory to backfilling outside the walls, were found to be as clean as when they were constructed.

3. Spillway. Dewatering efforts were minimal under the first contract (Dahlstrom), being limited to ditching at the overburden/shale contact along the right sideslope of the partially excavated spillway, the ditches emptying into the excavation where the spillway channel entered the flood plain. The method was not wholly effective as the ditches leaked. Also ground water seeped from the unditched entry slope at the south end of the excavation (upstream). Dewatering under the completion contract (Abrams) consisted of a more extensive system of ditching for gravity drainage. Two ditches protected the right side of the excavation from upstream beyond the right wall refill excavation downstream to approximately station 31+50 where the ditches joined. A single ditch continued downstream for a considerable distance beyond the junction of the two ditches. A temporary ditch was dug, then plugged with clay, along the face of the very rough cut backslope of the refill area, from about offset 260 feet left (north) to the vicinity of the centerline. This was to control flows of water into the refill area from incompletely excavated gravel in the approach area. Later, after most of the waterbearing gravel had been removed from the approach area, this barrier was replaced by an interceptor ditch within the approach area,

parallel to and upstream from the 1 on 1 refill backslope. Only a short length of interceptor ditch was needed to protect the left side of the approach and refill areas, as most of the ground water in the gravel on this side of the spillway had been dissipated earlier in the process of excavation. The stilling basin was not initially well protected from flooding by the San Gabriel River. A flood occurred before the left wall footings were complete and before the foundation for the stilling basin slab had been fine graded, causing river water to back into the area sufficiently to float tool sheds. After the floodwater receded, a temporary dike was built angularly across the spillway downstream near station 42+00, which doubled as a haul road for removing material excavated in the right side of the spillway. The 24-inch flap gate, which had been used in the discharge channel at station 72+00 under the first contract (Dahlstrom), was reinstalled in a pipe under the new dike/haul road.

Excavation for the refill at the left (north) side of the spillway was initially required to reach depths sufficient to remove all shale containing joints and fractures with weathered margins, not considering the proportion of the rock matrix which they constituted. The resulting depth of excavation was both variable and excessive. A decision was made to leave shale containing weathered joints and fractures in the foundation unless they constituted a significant proportion of the rock matrix. One result of excavation, as initially required, was production of wide, isolated, shallow depressions in the excavated surface of the shale. In order to prevent the depressed areas from collecting and retaining water percolating through the granular refill, particularly during construction, they were partially backfilled with select impervious (clay), the top surface of which was sloped toward and along a "notch" which had been cut to remove weathering along a fault crossing the refill area from the backslope to the chute slope. Thereby, any water which would subsequently percolate through the granular refill could be expected to encounter the sloping top of the backfill clay beneath, then flow to and along the fault "notch" into the chute drainage blanket for disposal. This construction practice was employed again near the right (south) si of the refill excavation and along a fault in the same general are .

Water seeped into each of the key and drain excavations in the stilling basin during their construction. This was in distinct contrast to experience in the keys and drains excavated in the chute slope earlier. Water entry into the deeper excavation for the end sill key (9 feet deep) was more rapid, flowing in some places, probably due to greater differential head resulting from the greater depth of the excavation. Sites of water emission from the foundation shale were generally easy to locate in this excavation, but were very difficult to locate in the shallow keys and drains. Seepage and flowage were easily controlled by pumping, but the excavations would fill with water if they were not pumped. Water seeping or flowing into the keys and drains of the stilling basin had not been expected as no free water had been

encountered previously, while excavating in the shale, and because the shale was considered to be impervious. The only known exception to this was in the key portion of the right (south) wall footings in the stilling basin where the foundation shale emitted small amounts of water (see plate 32). The source of moisture noted infrequently in key and drain excavation in the chute slope was the drainage blanket. The most reasonable explanation for the relatively large volume of water entering key and drain excavations in the stilling basin seemed to be that the principal source of water was rain percolating through the granular refill upstream. (The granular refill was not covered by a slab until late in the construction of the spillway). For this explanation to be valid, the water must have worked its way from the granular refill down slope to the drain at the foot of the chute, bypassing the intervening concrete keys. The drain system, in addition to the cross drains themselves, includes pipes angling downstream, breaching key concrete, which connect to the manholes outside the walls. Also, there are three rows of drain pipe sections, extending directly downstream from manholes in the cross drains under the chute, which discharge on the chute surface, penetrating key concrete enroute. These members of the spillway drainage system were not fully assembled until late in construction of the chute. The most credible evidence that the granular refill was the source of water which seeped and flowed into excavations for keys and drains in the stilling basin was found in the incomplete chute drain system. Water flowed from sections of the filter cloth-covered pipe in the cross drain at the foot of the chute slope (see figures 82 and 84). Also, water flowed from this drain pipe and silt resembling the silt fraction of the refill was found inside it when temporary plugs were removed from it. The cross drain pipe was flushed before of the drain system was completed. See plates 42, 43 and 44 for the spillway drainage system.

None of the water seeping into the drains and keys of the stilling basin, downstream from the bottom of the chute, was seen coming from the drainage blanket. So far as could be discerned, all seepage and flowage was through the shale foundation. The most surprising aspect of the seepage under the stilling basin was the slight head differences producing water movement through the shale over distances of from 20 to 40 feet between the drains. The bottom of the three drains are only 1.3 to 2.0 feet below the excavated surface of the shale on the bottom of the stilling basin, and the bottom of the two shallow keys are only 3.0 feet below the shale surface. The distance between the most downstream drain and the end sill key excavation was a minimum of 40 feet. Most of the flow from the upstream, or stilling basin side of the end sill key excavation, occurred within the shale from 4 to 5 feet below the adjacent shale foundation of the stilling basin slab. See figures 85, 86, and 87. Water also flowed from the shale of the downstream face of the end sill key at a few places. Water also entered the key excavation from above the foundation through compacted shale debris, used as an access road, immediately downstream from and parallel to the end sill key. The water which flowed from the downstream face of the end sill key, that

entering through the compacted shale debris downstream, and possibly a small amount of earlier seepage into keys and drains appeared to have come from a large pond of surface water about 150 feet downstream from the stilling basin. See figures 88 and 89 for a view of this area.

The apparent ease with which water seeped or flowed through the foundation shale into excavations in the floor of the stilling basin, and the varying depths at which it occurred, suggest that water was transmitted through the numerous partings in the shale. The tendency of the shale to part probably originated at or before the time of faulting, caused by deformational forces which led to faulting. Likely the partings became effective seepage and flowage paths when they opened as a result of both unloading by regional erosion and by extensive spillway excavation. Conceivably, there may be a greater number of seepage pathways in the uppermost 1 to 2 feet of foundation shale because of additional disruption from the use of scraping tools to excavate. However, this has not been established.

The slab cross drains, as designed, caused construction problems. The slab cross drains are effectively strips of abnormally thick drainage blanket, located along each concrete key on the upstream side, through which extends a perforated, filter covered, plastic drain pipe. The principal construction problem was how to maintain cross drain filter sand on a slope of 2 vertical on 1 horizontal, the slope being 2.5 feet to 3.0 feet high. The slope must be maintained long enough to allow excavation of the conterminous key, placing reinforcement in the key, and placing key concrete. Early experience in key construction was that the drain sand slumped into the key excavation to varying degrees, even though the sand had been moistened, frequently exposing the cross drain pipe, allowing key concrete to reach the drain pipe. See figures 82 and 83. This also resulted in development of a large void under the work slab covering the cross drain, which required removal and later replacement of that portion of the work slab.

Solution of this construction problem by the alternative procedure of forming the upstream face of the key concrete in a key/drain excavation without sand was not available because of the requirement that the foundation shale be protected from dessication. Protection in this instance meant moist drain sand. The only solution available to maintain drain sand in proper position while placing reinforcement and subsequently placing key concrete was use of some type of permanent forming. Forming between the drain sand and the key concrete could only be some type of sheet material, such as plywood or sheet metal. This forming had to remain after concrete placement because of the shape of the reinforcement oriented up and downstream. These reinforcement bars are shaped in such a way that their ends protrude both up and downstream, producing an overhang, which effectively prevented withdrawal of any kind of flat stock forming between the drain sand and the key concrete. Fortunately the contractor (Clearwater Constructors) had a large supply of surplus, though corrugated, sheet metal which he

was willing to use as forming and to leave in-place. Design of slab cross drains, such as those in the spillway chute at Granger Dam, can be materially improved by moving the drains upstream sufficiently that the sand on the downstream edge of each drain is no thicker than the drainage blanket itself. While this does not eliminate the problem of sand retention, it does reduce it considerably. Other design problems involving the drainage blanket will be discussed subsequently under FOUNDATION ANCHORS.

FOUNDATION PRESERVATION

1. General. Foundation preservation, as used here, refers primarily to protection of the shale foundation from desiccation. The single specification applying to foundation preservation in the cutoff trench was that the trench was not to remain open for more than 60 days. This was only under the final contract. Required foundation preservation in the outlet works excavation was to consist of leaving 1 foot of shale above final grade of the "flat" until ready to backfill the excavation, and to covering the structure foundations with 6 inches of protective concrete where shown in the plans. Foundation preservation in the spillway, as specified and as shown in the plans, involved limiting exposure to drying to a maximum of 4 hours for the wall footings and the slab foundation, covering horizontal surfaces with 4 inches of protective concrete, and by covering vertical and inclined surfaces with 1-1/2 inches of pneumatic concrete. Covering material for the slab foundation and drains was to be the drainage blanket (sand was always delivered moist and its bottom remained moist for as long as needed in every instance). Wet matting shale surfaces (well moistened concrete curing mats), covering with Aerospray 52 Binder, and foundation flooding were other methods used to protect the spillway foundation from dessication.

2. Outlet Works. Foundations of the approach slab and gate tower were protected by 6 inches of protective concrete. As indicated earlier, the modified foundation of the conduit was protected by either backfill concrete or protective concrete, depending upon location. The contractor (Hensel Phelps) placed a protective concrete slab beneath the conduit headwall to facilitate use of a "sand box" to support the conduit invert form where the conduit extends midway into the headwall (prior to headwall construction). The foundation of the chute and stilling basin was protected by moist sand of the drainage blanket. Protective concrete was placed in the narrow excavation surrounding the large block of shale left in place under the end sill, though this was not required by the plans.

3. Spillway. Protective concrete (4 inches thick) was required to cover the impervious portion of the refill beneath the wall footings from their upstream ends downstream to the cutoff key at station 28+88.27. Protective concrete was also used under the wall footings from commencement of the shale foundation downstream to the keys under the ends of the walls in the stilling basin. The stilling basin slab foundation between 32+95.27, the end of the drainage blanket, and the backslope of the end sill key at station 33+10.71 was also protected by 4 inches of protective conrete. The vertical and steeply inclined (12 vertical on 1 horizontal) shale slopes along the outside of the left wall footings were protected by the application of 1-1/2 inches of pneumatic concrete. The pneumatic concrete stopped at the top of the slope where it meets the 1 vertical on 3 horizontal spillway sideslope. Some of this pneumatic concrete cracked and portions of it separated from the shale surface against which it had been applied. Pieces of

pneumatic concrete fell from the slope in a few places. This experien prompted modifying the method of applying pneumatic concrete to strengthen and better hold the concrete against the foundation shale o the right wall footing sideslopes. The modification required use of heavy wire mesh (2X2X14/14 welded wire fabric) over which the pneumati concrete was to be placed, the wire mesh being supported by L-shaped lengths of No. 4 reinforcing bar (1/2 inch), driven through 3/8 inch holes drilled 2 feet into the shale on 5 foot centers (horizontally). See figure No. 90 for the modification drawing. Shortly after commencing this work, support for the mesh was changed from No. 4 reinforcing bar to bridge nails 1 foot long inserted into drilled holes an driven the final 2 inches into the shale. Weep holes 1-1/2 inch in diameter were required to be drilled or formed through the pneumatic concrete on 10-foot centers. A rollover section was required at the t of the slope partly to "hang" the pneumatic concrete and partly to increase the distance to the nearest drying surface, which was the unprotected, 1 vertical on 3 horizontal spillway sideslope above, and also to reduce access of rain to the top of the pneumatic concrete on the slope. See figures 72, 73, 74, and 75 for view of the upstream po tion of the installation. However, the only qualification which the shale was expected to have was that it should not exert positive earth pressure against the wall footings (which it did not). Consequently, this protection was provided for the shale on the outside slopes of th wall footings only because of their function as a forming surface for struct 1 concrete of the wall footings.

As described earlier under EXCAVATION PROCEDURES, the foundation f the chute slab was fine graded in strips extending down the chute slop The finished shale surface of each strip was protected from dessicatio by covering it with moist drainage blanket sand. On completion of exc vation of each strip, moist sand was spread down the slope, usually by Caterpillar D-3 dozer having extra-wide tracks. See figures 67 and 68 The stilling basin slab foundation, upstream from station 32+95.27, w also protected by spreading moist drainage blanket sand.

The keys were all excavated commencing at the left (north) wall, working toward the right (south) wall. All chute keys in which the foundation is shale, rather than refill, were protected for 1 day construction cycles by means of covering the foundation with well moistened concrete curing mats. The procedure was to excavate a convenient length of key one day (usually in lengths equal to the width c one slab panel, which is 30 feet (or in multiples of this length); prc tect the excavation overnight by wet-matting; then bulkhead off the following morning and set sectional units of prefabricated reinforcin steel; then place concrete while excavating the next section of the ke All of the keys and drains of the stilling basin upstream from the en sill key were protected from desiccation by involuntary flooding. As described earlier under DEWATERING AND FOUNDATION DRAINAGE, water see into each of these excavations during their construction, automatical protecting them from desiccation. The end sill key was excavated for

distance of slightly more than 12.5 feet inward from each of the walls immediately after completion of the wall footings (through the 12-foot wide slab closure lanes). These sections of key excavation were not to receive reinforcement or concrete until later, and their shale surfaces were protected by two methods. Pneumatic concrete was applied to the shale, but only the upper portion of the sideslopes received the required thickness of concrete because of the profile of the key excavation. Since the end sill key is 9 feet deep, 1.5 feet wide at the bottom, and 4.5 feet wide at the top, the nozzle used to apply pneumatic concrete could only approach a 90 degree relationship with the key sideslopes along the upper part of the slopes where the desired thickness and quality of application could be maintained. Use of a right angle nozzle in such a narrow and deep excavation was not expected to produce improved results. Shortly after excavation and protecting the key with pneumatic concrete the 12.5 feet stub key sections filled with seep water and remained so until further key construction began, giving the foundation shale additional protection from desiccation. The progressively deeper wall keys, which commence at the outer edge of the downstream end of both the left and right wall footings and wrap around the ends of each wall footing to become the end sill key at station 33+15.27, were constructed on 1 day construction cycles and were protected by wet concrete curing mats. Alternative means to protect the shale surfaces of the end sill key excavation were considered as a result of adverse experience with unsupported pneumatic concrete along the outside slopes of the left wall footings. Wire mesh-supported pneumatic concrete was considered. But, for reasons of economy, it was decided to test a spray product called Aerospray 52 Binder[2] in the first 200 feet of the end sill key inward from offset 462.5 feet left (inward from the 12.5 stub key section). If protection by this material proved adequate, it would be substituted for pneumatic concrete in the remainder of the end sill key. The only test required for acceptance of Aerospray 52 was visual inspection. In the field, it was decided that quantitative documentation by means of moisture samples would be most desirable. Chip samples of shale were taken from a sideslope of the key and collected in a plastic pail on completion of each section of the key. All samples were taken to the contractor's soil laboratory (Abrams) for moisture determination. A second sample was taken immediately adjacent to, and from the same stratum as the initial sample, just before placement of concrete, first removing the Aerospray film. The end sill key was constructed on a 1 day cycle as described earlier, except for an occasional weekend when the key remained open to the air and seepage flooding. Visually, the Aerospray 52 proved adequate as essentially no desiccation checking or cracking was found.

---

[2]Aerospray 52 Binder is a dispersable alkyd resin product (which polymerizes) of American Cyanamid Co., Wayne, N.J. 07470, which had been used previously by the Corps of Engineers.

Moisture samples from the 200 feet test section showed the moisture content of the shale to change only 0.5 to 0.6 percent, even over a weekend (0.6 percent). The use of Aerospray 52 was approved for the remainder of the end sill key excavation. Sampling for moisture content data was continued until the key was completed to make additional records on this product. Moisture content data from all tests are compiled in Table 1 which follows. Aerospray 52, as used by the contractor, was diluted with water in a ratio of 1:1. It was applied using a hand-pump garden sprayer. Best results seemed to be obtained by adding spray concentrate to water for mixing rather than the converse, which usually produced poor mixing results and clogging of the hose and nozzle. Based on experience, Aerospray 52 diluted 1:1 and applied in a coarse spray, should not be depended upon to protect calcareous shale, such as that of the Taylor Group, for longer than a weekend without respraying, especially during hot weather. Aerospray 52 did not exhibit good adhering ability when exposed to rain. The spray film on test panels of shale usually broke loose from the shale producing the appearance of rippled skin. Also, the resulting surface film did not seem an adequate cover for areas subject to traffic.

TABLE I

SHALE SAMPLE MOISTURES BEFORE AND AFTER USE OF AEROSPRAY BINDER 52 ™

IN END SILL KEY EXCAVATION, STATION 33+15.27

Sample No.	Date	Time (Hrs)	Location of Sample	Moisture Content (%)	Moisture Loss (%)
K-100	3-16-78	1658	430' Lt, El 436.9± (3.6' above bottom) Upstream Slope	15.9	
K-100A	3-17-78	1057	429' Lt, El 436.9± (3.6' above bottom) Upstream Slope	15.4	0.5
K-101			No Sample		
K-102	3-20-78	1527	369.8' Lt, El 436.5± (3.7' above bottom) Upstream Slope	17.2	
K-102A	3-21-78	1046	369.2' Lt, El 436.8± (4.0' above bottom) Upstream Slope	17.2	0.0
K-103	3-20-78	1620	342.5' Lt, El 437.8± (4.3' above bottom) Upstream Slope	16.5	
K-103-A	3-21-78	1108	341.6' Lt, El 437.9± (4.4' above bottom) Upstream Slope	16.0	0.[illegible]
K-104	5-23-78	1630	280.1' Lt, El 439.9± (3.6' below top of protective concrete slab and 2.3'/2.5' above Shale I/II contact) Downstream Slope	17.1	

TABLE I

SHALE SAMPLE MOISTURES BEFORE AND AFTER USE OF AEROSPRAY BINDER 52 tm

IN END SILL KEY EXCAVATION, STATION 33+15.27

Sample No.	Date	Time (Hrs)	Location of Sample	Moisture Content (%)	Moisture Loss (%)
K-104A	5-25-78	1435	277.9' Lt, El 439.9± (2.4' above Shale I/II contact) Downstream Slope	16.5	0.6
K-105	5-25-78	1449	252.5' Lt, El 441.2± (2.1'/2.2' above Shale I/II contact) Downstream Slope	16.8	
K-105A	5-26-78	1045	251.5' Lt, El 441.2± Downstream Slope	16.8	0.0
K-106	5-25-78	1212	159.5' Lt, El 438.8± (5.3'above bottom) Upstream Slope	17.8	
K-106A	5-30-78	1000	161.1' Lt, El 438.8± Upstream Slope	16.7	1.10
K-106B	5-31-78	1037	158.5 Lt, El 438.8±	15.5	2.3 % difference from K-106 1.2% difference from K-106A
K-107	5-30-78	1640	102.1' Lt, El 439.1± (6.6' above bottom) Upstream Slope	17.2	
K-107A			No Comparison Sample		
K-108	5-31-78	1550	40.1' Lt, El 439.8± (6.5' above bottom) Downstream Slope	18.0	
K-108A	6-02-78	1330	37.1' Lt, El 439.8± Downstream Slope	17.9	0.1

TABLE I

SHALE SAMPLE MOISTURES BEFORE AND AFTER USE OF AEROSPRAY BINDER 52 tm

IN END SILL KEY EXCAVATION, STATION 33+15.27

Sample No.	Date	Time (Hrs)	Location of Sample	Moisture Content (%)	Moisture Loss (%)
K-109	6-02-78	1400	3.5' Lt, El 439.1± (5.7' above bottom) Downstream Slope	18.8	
	NOTE: Key flooded with water 6-03-78 to 6-09-78				
K-109A	6-09-78	1515	2.7' Lt, El 439.1± Downstream Slope	17.8	1.0
K-110	6-27-78	1520	52.5' Rt, El 438.8± (5.5' above bottom) Upstream Slope	17.9	
K-110A	6-28-78	1120	Composite Sample: 53.7' Rt and 51.5" Rt, El 438.8± Upstream Slope	17.1	0.8
K-111	6-28-78	1720	198.5' Rt, El 438.3± (4.7' above bottom) Downstream Slope	17.7 Strong petroleum odor	
K-111A		Unable to sample due to surface water flowing over sample site.			
K-112	7-05-78	1640	354.5' Rt, El 438.7 to El 439.5 (5.4' to 6.2" above bottom) Downstream Slope	20.4	
	NOTE: Key flooded with water 7-05-78 to 7-26-78				
K-112A	7-26-78	1200	355.5' Rt, El 438.7 to El 439.5 Downstream Slope	22.0	Moisture increase 1.6
K-113	7-26-78	1210	446.0' Rt, El 437.9± Sampled immediately above local seam, near fault - Downstream Slope	22.3	
K-113A	7-27-78	1007	446.6' Rt, El 437.9± Downstream Slope	20.2	2.1

FOUNDATION MAPPING METHODS

1. Cutoff Trench.

a. Initial Contract. In the following discussion, the inspection trenches from station 1+90 to the spillway and from station 147+50 to station 159+00 are considered part of the cutoff trench. All mapping in the cutoff trench under the initial contract was done by plane table surveying. The project geologist acted as rodman and the plane table was operated by a Corps surveyor. Following approval of each section of trench between the spillway and approximately station 57+70, the trench was mapped first on the bottom and up the lowermost sideslopes for a vertical distance of 5 to 10 feet above the bottom to allow backfilling to commence as soon as possible. The remainder of the sideslopes were mapped subsequently. Most of the upper portion of the sideslopes were mapped before the trench bottom was finished between approximately station 57+70 and the end of the trench at station 60+48, at the contractor's request. The trench bottom was then completed, approved, and mapped in short segments as previously described under EXCAVATION PROCEDURES. Some difficulty was experienced in mapping these segments because of slope plating. Questions on pay quantities of excavation and backfill in the right abutment derived from the contractor's profiles, which consisted only of shots at the top and at the toe of the slopes, were ultimately resolved by means of trench plane table data and photographs of trench excavation. Geologic mapping of the cutoff trench was done after shallow trenching the sideslopes and/or spot checking with a mattock (grub hoe). Test digging also provided samples for visual and tactile classification of overburden materials. Too often the sideslopes appeared to be clean, but were smeared or covered with a thin mantle of moved and compacted material, which could have caused erroneous mapping. The overburden comprising the sideslopes between station 57+70 and station 60+48 was particularly difficult to map in places where the contractor's dragline had been used.

b. Final Contract. With only a few local exceptions, the cutoff trench from its north end in the left abutment at station 159+00 south to the closure section was mapped from data collected on profile stations 100 feet apart (such as 76+00, 77+00, 78+00, etc.) and from sketches or interpretation between profiles. Geological data were collected along surveyed profiles, obtained by measuring slope distance from geologic changes to survey hubs and stakes, toe of slope, or top of slope. Data were collected concurrently with the contractor's surveying of quantity profiles; however, some of the data from the upper slopes were obtained earlier. The location of features to be mapped or noted on the trench bottom was usually determined by the contractor's survey party during approval, but was occasionally obtained by taping. Surveying and geologic mapping in the cutoff trench where Willis Creek channel crosses the trench was considerably enhanced by 108 stadia shots taken by the contractor's survey party, 41 of which were principally for geology, the remainder for topography of the trench. Slope distance

data from surveyed profiles were scaled on plotted profiles having their vertical scale equal to their horizontal scale. Offsets of geologic changes and offsets of topographic contours measured on the plotted profiles, were used for map construction. Mapping by plane table is preferred over mapping from profiles with help from sketches as plane table shots can be located where needed with little interpretation or estimation necessary between shots.

2. Outlet Works. Geologic and topographic mapping in the outlet excavation was done initially with a plane table. The right 1 vertical on 3 horizontal overburden slope was mapped with a plane table from station 10+20 to station 18+00, though there was minor modification of the slope between approximately station 16+50 and station 18+00 later. A plane table was used to map on the left overburden slope between station 15+50 and station 18+00, and on the conduit foundation between station 13+60 and station 15+05. Significant errors in the subcontractor's profiles (Haufler) on the outer 1 on 3 overburden slopes were found with the plane table, causing a resurvey of these slopes by the subcontractor. Portions of the extensive "flat" area adjoining the conduit were mapped by plane table, with the project geologist as rodman and an engineering technician operating the plane table. Much of the remainder of the "flat" area was mapped with a plane table by the project geologist using a transit tripod to support the rod. The overburden sideslopes (upper 1 on 3 slopes) upstream from station 10+20 on the right side and upstream from station 15+50 on the left side were mapped by measuring slope distances to geologic changes and plotting on contractor profiles on a natural scale (1:1) to obtain mapping offsets for geology and topography. All foundation checking and mapping in the excavations for the approach structure, gate tower, conduit, chute and stilling basin, excepting between station 13+60 and station 15+05 where a planetable was used, was done by tape and surveying level from contractor reference marks. Geological data outside the walls of the chute and stilling basin were also obtained by this method. A total of 35 stadia shots were made to trace faults and the lower depositional seam in the steep left shale slope outside the chute and stilling basin during the final contract, by the contractor's survey party (Abrams). Mapping of the steep, uppermost, right shale slopes outside the chute and stilling basin walls was minimal due to steepness of the slope (safety). Minor trimming of the uppermost shale slope outside the stilling basin walls was necessary as was grading the right and left berms after the slope was surveyed, requiring modification of the mapping.

3. Spillway. Geology and topography portrayed on the accompanying foundation map, plates 32 and 33, were drawn from profile shots, and geology shots on the outer slopes, in the refill area, on the slopes of the chute, and on the flat bottom of the stilling basin. Geologic features in the shale of the wall footings, keys and drains were located by tape and level from contractor reference marks. Profile and geology shots were made by the contractor's survey party. The profiles were final profiles for pay quantities and the geology shots were made as a

voluntary extra part of that work by the contractor's survey party. A large number of voluntary stadia shots for geology and topography were also made along the top of the refill backslope and on the right sideslopes.

FOUNDATION ANCHORS

1. Installation Procedure and Testing. Installation of an anchor involves drilling and cleaning a proper sized hole to the required depth, preparing and installing the anchor in the hole, and backfilling the hole with grout. Preparation for drilling anchor holes commenced prior to construction of the work slab by placing lengths of stovepipe or other thin sheet metal conductor pipe at the sites of and in the orientation required for the anchor holes. Anchors in both the chute and stilling basin of the outlet works were oriented vertically. Anchors in the spillway were oriented at 90 degrees to the surface of the slab. The thin conductor pipe was cut sufficiently long to reach from its embedment in the top of the shale foundation upward through the drainage blanket and the work slab. The anchor holes were drilled through the conductor pipe, using compressed air for removing the cuttings, the drill being situated on the work slab. After drilling and blowing the anchors holes clean, the previously prepared anchors were suspended in the holes, ready for the holes to be backfilled with grout. Anchor preparation in the spillway consisted of painting the upper 5 feet of the precut and formed No. 11 bar anchors with bituminous paint, wrapping the painted section with bituminous impregnated paper, and adding the "chairs" below the wrapped section. Preparation of No. 14 bar anchors in the spillway was the same except that the bituminous section was 10 feet long rather than 5 feet. The "chairs" were actually centralizers, in drilling parlance, and those used differed from those shown in the plans. The "chairs" consisted of three hump-shaped lengths of rod stock, oriented parallel to the anchor, spaced at 120 degrees around the anchor, and welded to a half-ring of the same rod stock at the top and the bottom of the "humps." The "hump" and half-rings are welded to form prefabricated units, which are slipped on the anchors at the appropriate position and wired in place. No specific preparation (such as wrapping) was required for anchors in the outlet works. The final step in anchor completion was backfilling with grout. In this operation, a grout hose was extended past the centralizers nearly to the bottom of the holes initially, and was withdrawn as the holes filled with grout. Five pullout tests were conducted in the spillway under the direction of the Structural Section prior to cutting anchors to length and drilling anchor holes, three tests on No. 11 bar anchors, and two tests on No. 14 bar anchors. Results of the tests confirmed the adequacy of the anchor lengths and hole depths specified for the spillway. No pullout tests were made in the outlet works. Voids were discovered in the near surface grout of two completed anchors during inspection of a key on the spillway chute slope. Appearance of the voids suggested either that the grout hose had been withdrawn faster than the grout had filled the holes or that the grout hose had been withdrawn from the hole and the hole backfilled from the surface with stiff grout. As a consequence of this discovery the contractor tested a large number of completed anchors with a small mobile crane in the area of the defective anchors. Calibration and loading of his tests were inadequate. Following the contractor's tests, pullout tests were required to be con-

ducted on five completed anchors, selected by the District Office, two tests on No. 11 bar anchors and three tests on No. 14 bar anchors. All of these tests produced satisfactory results.

The most prevalent problem experienced in anchor construction was adequacy of seating of the conductor pipe in the foundation shale. A significant number of instances of inadequate seating were found in the spillway. Once the problem was recognized, the remedy, in the form of embedding the bottom of the conductor pipe in the shale and placing a sealing ring of grout around the pipe, was added to preparation procedure by the contractor. Inadequate seating of the conductor pipe allowed drainage blanket sand to be entrained in the return flow of air during drilling, producing voids under the work slab. Since lost drainage blanket sand could not be replaced without removing large pieces of the work slab, the voids were filled with grout, gently introduced through holes in the work slab. One of the better but simpler tests for voids beneath the work slab was tapping the slab with a wood pick handle, listening for a hollow sound.

2. Problems. The finding of voids beneath the spillway work slab suggests a possible problem for the Granger structures and similar concrete structures where both drainage blankets and anchors are used. The spillway slab and the U-frame chute/stilling basin of the outlet works are supported by the drainage blankets beneath them. Their anchors are embedded in (supported by) the shale foundation. Because there is a significant risk of either minor settlement of the drainage blanket or of loss of its sand during construction of keys and anchors, there is a significant risk of transfer of load of the concrete structures from the drainage blanket to the anchors, in effect making piers of them. The magnitude of settlement or loss of drainage blanket sand necessary to cause load transfer is negligible. Fortunately these concrete structures are not normally considered a heavy loading. A different drain design should be required for future structure foundations if the structures and their anchors are to be founded on and in the same material.

FOUNDATION INSTRUMENTATION

The following types and numbers of instruments were installed to obtain foundation performance data:

Type of Instrument	Number of Instruments Installed	Number of Instruments Read Periodically Beyond Construction Period
Piezometers	69	61, Quarterly
Settlement Plates	10	9, Semi-annually
Outlet Works Conduit Reference Marks	35	35, Semi-annually
Spillway Heave Gages	5	0 (Destroyed)
Slope Indicators	9	0 (Destroyed)

Foundation performance, as interpreted from instrumentation readings, is reported in the Embankment Criteria and Performance Report.

KNOWN PROBLEMS

1. Spillway. The right (south) sideslope of the approach channel near the concrete structure is expected to continue emitting ground water. The source of the water is the gravel and sand of the extensive upland terrace bordering the San Gabriel River valley to the south. Seepage will probably keep the bottom of the approach area soft and wet for at least 1,000 feet upstream from the spillway structure unless it is captured and diverted.

POTENTIAL PROBLEMS

1. Spillway. The refill, on which the upper part of the chute, the weir, and the approach are founded, is comprised of two materials; granular fill and clay fill. Horizontally, the change from granular fill to clay occurs at the approach cutoff key at the upstream end of the slab. A problem of differential settlement beneath the walls in this vicinity may exist. Evidence of this, should it occur, would probably be distress at wall monolith waterstops.

The finding of silt in pipes of the drainage system during construction suggests the possibility of future plugging of the system by migration of additional silt from the granular refill into the cross drains. Should plugging occur, the only visible evidence of it would likely be seepage from construction joints along the keys of the chute slope. No such visible evidence can be expected in the stilling basin as the stilling basin will remain full of water at all times. The remedy for this would be flushing silt from the cross drains to the manholes where it can be removed.

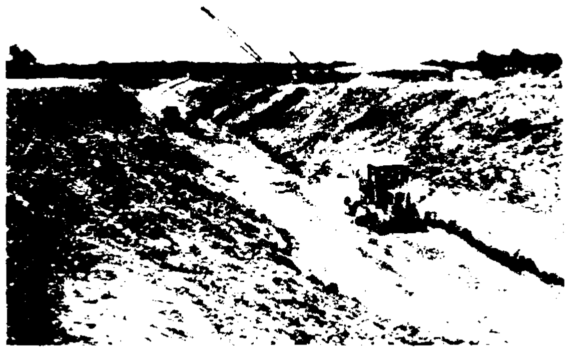


Figure 1. Cutoff trench, 1st contract: Looking north toward San Gat River from vicinity of station 41+00, upstream.

Figure 2. Cutoff trench, 1st contract: View of slopes cleanec dragline, looking downstream (right) in vicinity of station 43+00.

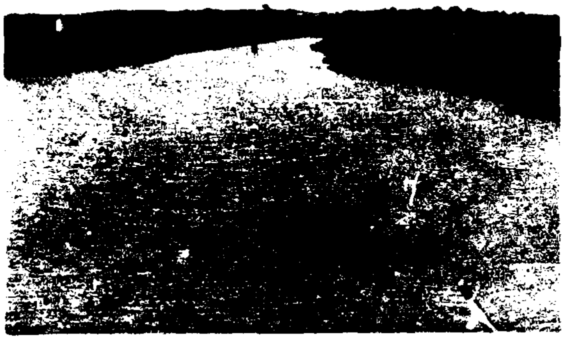


Figure 3. Cutoff trench, 1st contract: Looking toward right (south) abutment. *Flooding on morning following figure 1.* (Dike had been overtopped.)

Figure 4. Cutoff trench, 1st contract: Casing of large diameter auger boring.

Figure 5. Cutoff trench, 1st contract: Backfilling (near) and sealing off water with fill (distant) along trench toes. Casing of large diameter auger boring in distance.

Figure 6. Cutoff trench, 1st contract: Looking toward right (south) abutment from vicinity of station 59+50, upstream.

Figure 7. Cutoff trench, 1st contract: Looking toward dike (northward) from near station 56+50, upstream.

Figure 8. Cutoff trench, 1st contract: Looking toward dike from haul road at about station 58+00, upstream prior to clean-up of bottom.

Figure 9. Cutoff trench, 1st contract: Looking downstream across trench. Planetable is at station 59+00, 65 feet left, and dragline is at about station 59+25, downstream.

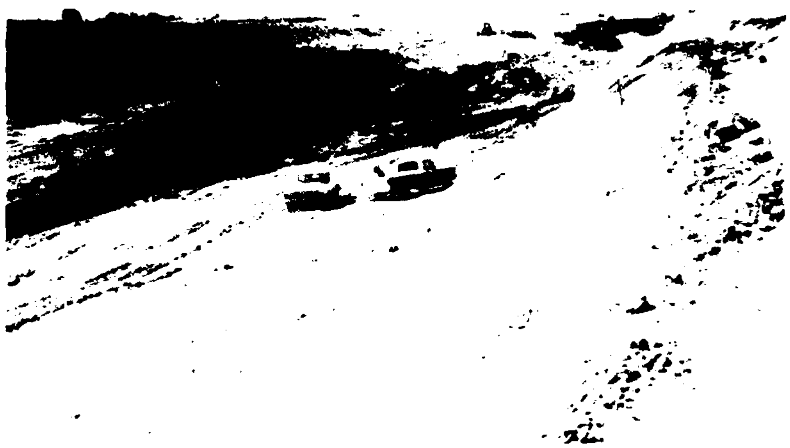


Figure 10. Cutoff trench, 3d contract: Looking north toward outlet tower and left abutment from about station 65+75, downstream. Balcones-type fault extends from station 67+65 (opposite side) to station 71+10 (this side).

Figure 11. Cutof
fault in slope at st
fault.

Figure 11. Cutoff trench, 3d contract: Water control ditch upstream, fault in slope at station 67+65, lower depositional seam present north of fault.

stream,
orth of

Figure 12.
section. 1s
excavatio
distant sl

Figure 12. Cutoff trench, 3d contract: Looking south through closure section. 1st contract embankment and cutoff trench fill at distant end of excavation. Filled San Gabriel River channel crosses view short of tank on distant slope.

closure
t end of
tank on

Figure 13. Er
overtopping of

Figure 13. Embankment, 3d contract: "Lake Sorefinger" following overtopping of upstream fill. Cutoff trench in left, upper part of view.

llowing
of view.

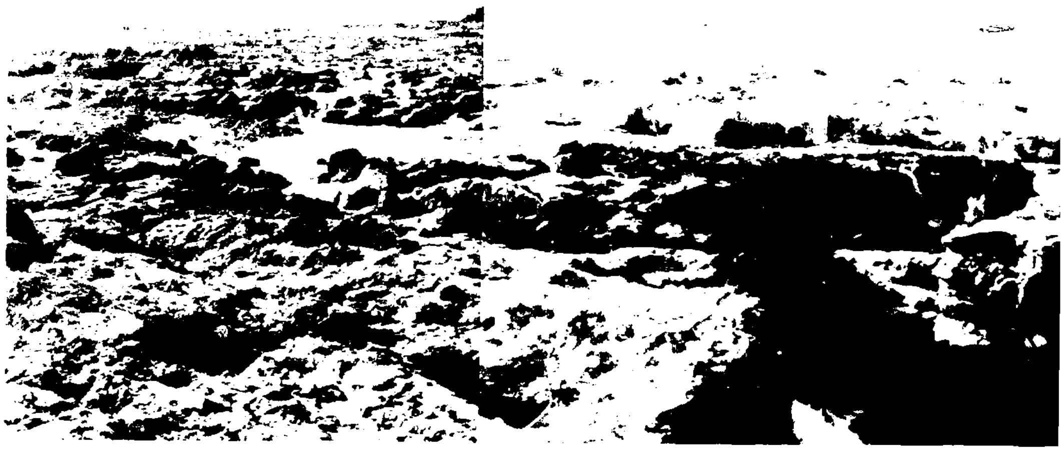


Figure 14. Embankment, 3d contract. Random zone material (upstrean overtopped by Sorefinger Creek. "Lake Sorefinger" being drained her Shale pile and piezometer here are also shown in figure 13.

Figure 15. Embankment, 3d contract: One strand line of "Lake Sorefinger" on semi-compacted zone material. Shale pile here is also shown in figure 13.

Fi
ax
cu
to
pc

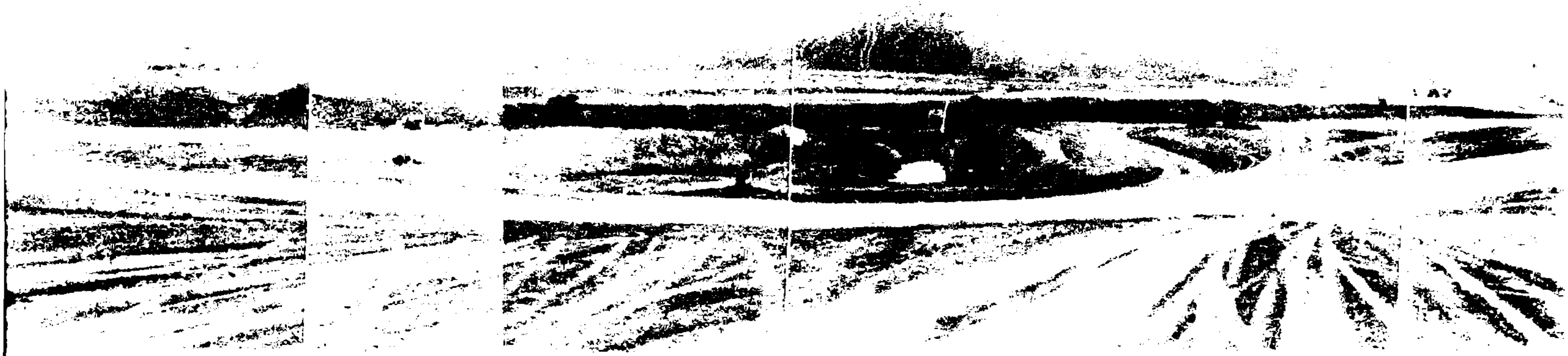


Figure 16. Embankment, 3d contract: View looking south along dam axis toward embankment of 1st contract. A short, incomplete section of cutoff trench is visible. San Gabriel River channel crosses view from right to left. Cleaning and backfilling of channel has commenced in right portion of view.

2

long dam
section of
from right
d in right

3

Figure 17. Embankment, 3d contract: Cleaning and backfilling San Gabriel River channel for closure. Refer to text for description of these operations.

Figure 18. Embankment, 3d contract: San Gabriel River channel looking upstream from near downstream haul road.

Figure 19. Outlet works, 2nd contract: Scallop-shaped, cuspate or conchoidal fracture and slip-block in backhoe trench for reexploration of the conduit foundation.

Figure 20. Outlet works, 2nd contract: Lower depositional seam, large scallop-shaped or cuspate fracture and pull out at station 13+93, 114 feet right. Looking downstream. Fault not visible.

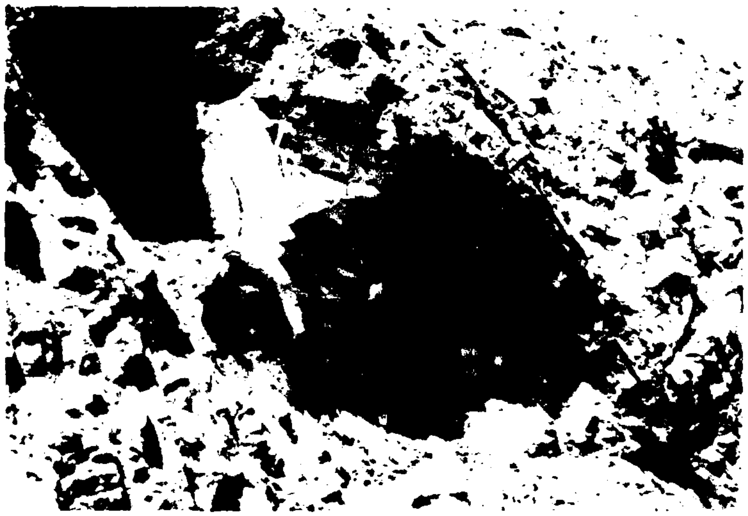


Figure 21. Outlet works, 2nd contract: Fault in slope at station 13+93, 114 feet right. Looking upstream.

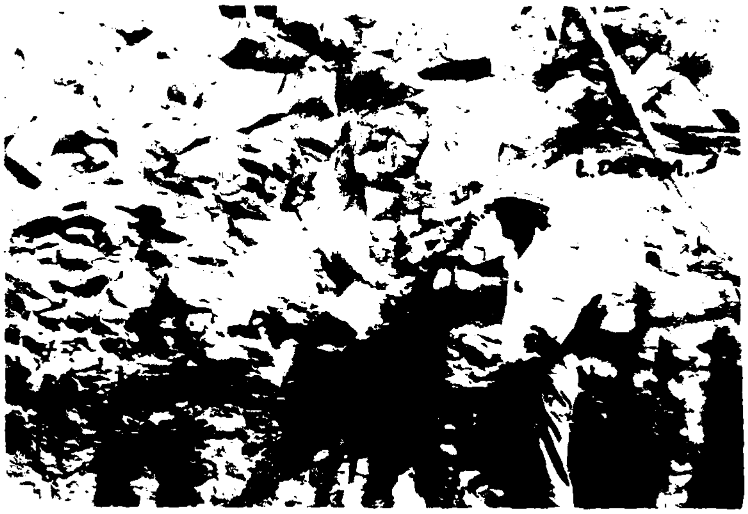


Figure 22. Outlet works, 2nd contract: Lower depositional seam and fault at station 14+52, left (north) sideslope of conduit excavation.

Figure 23. Outlet works, 2nd contract: Lower depositional seam, fault, and cuspate fracture at station 14+41, right (south) side of conduit excavation.

Figure 24. Outlet works, 2nd contract: Lower depositional seam and fault at station 8+35, right (south) sideslope of conduit excavation.

Figure 25. Outlet works, 2nd contract: View of conduit, looking downstream, during early excavation. Chute and stilling basin excavation not yet started.

Figure 26. Outlet works, 2nd contract: View of single monolith of conduit backfill concrete.

Figure 27. Outlet works, 2nd contract: Looking downstream. Conduit backfill concrete (near), placing invert structural concrete and forming for arch section (distant).

Figure 28. Outlet works, 2nd contract: Looking upstream. Conduit invert reinforcing (near), backfill concrete, and early tower construction (distant).

Figure 29. Outlet works, 2nd contract: Looking downstream and to right (southeast). Work area in the "flat", conduit invert, arch concrete started, chute and stilling basin partially excavated.

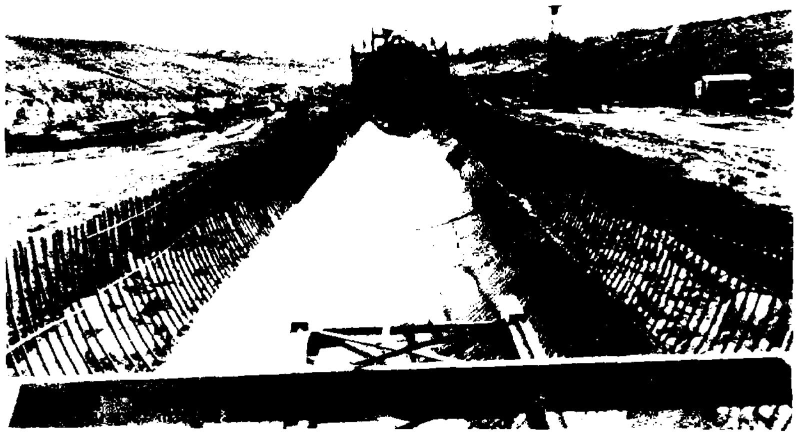


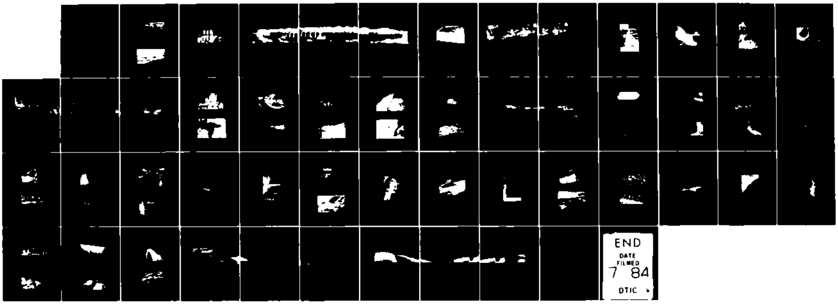
Figure 30. Outlet works, 2nd contract: Looking downstream. Completed invert section (near) and arch forms (distant).

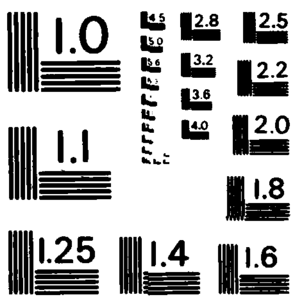
Figure 31. Outlet works, 2nd contract: Looking [illegible] (southwest). Work area in the "flat", conduit [illegible] construction (distant).

AD-A140 987 GRANGER LAKE EMBANKMENT-OUTLET WORKS-SPILLWAY VOLUME 1 (U) ARMY ENGINEER DISTRICT FORT WORTH TX G M RUEDE AUG 83

2/2

UNCLASSIFIED F/G 13/2 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

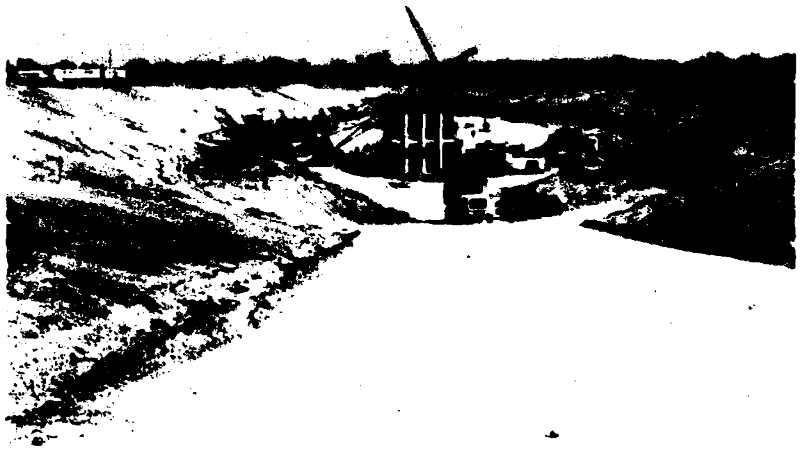


Figure 32. Outlet works, 2nd contract: Looking downstream toward tower along centerline of outlet. Approach structure not yet commenced.

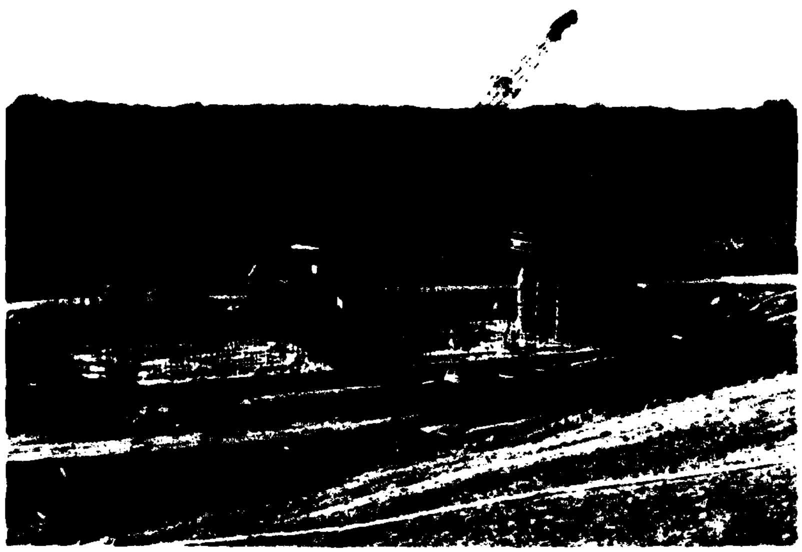


Figure 33. Outlet works, 2nd contract: Looking from left (north) to right (south) across tower slab.

Figure 34. Outlet works, 2nd contract: Looking downstream (east) through tower and transition construction.

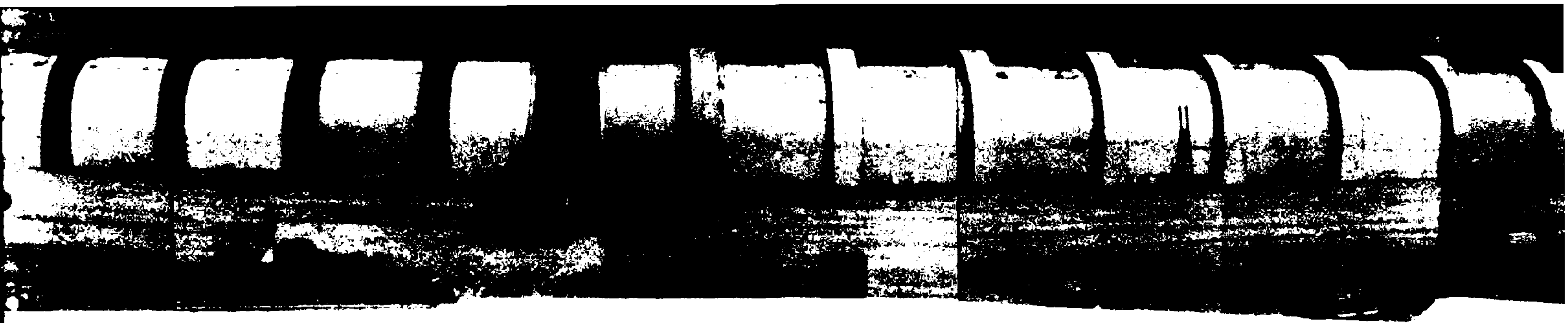
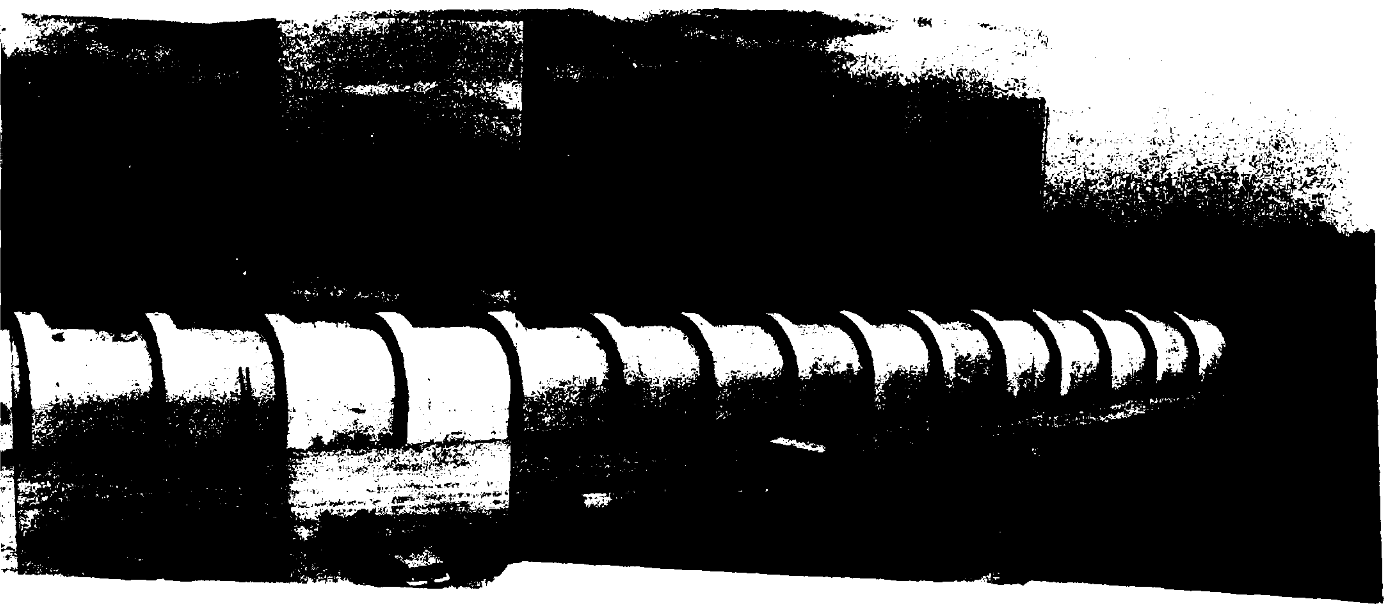


Figure 35. Outlet works, 2nd contract: View of tower and conduit, looking from right (south) to left (north).

2



ıd conduit,

3

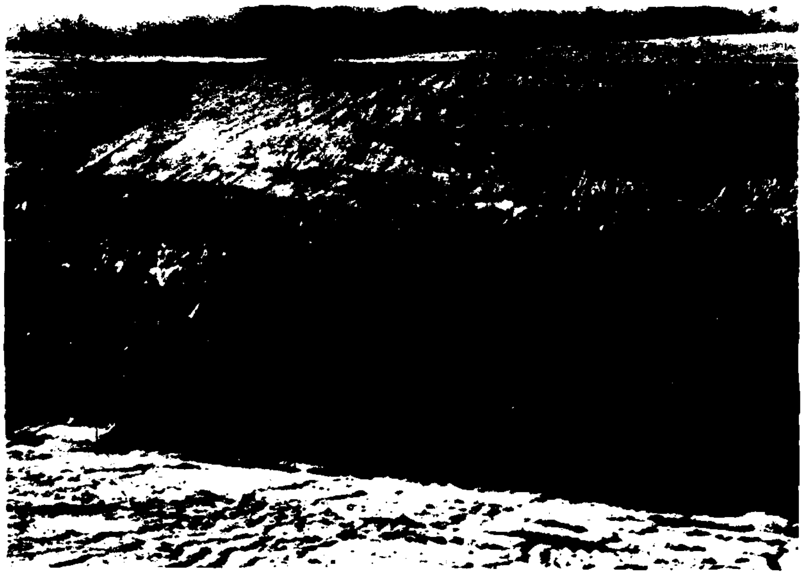
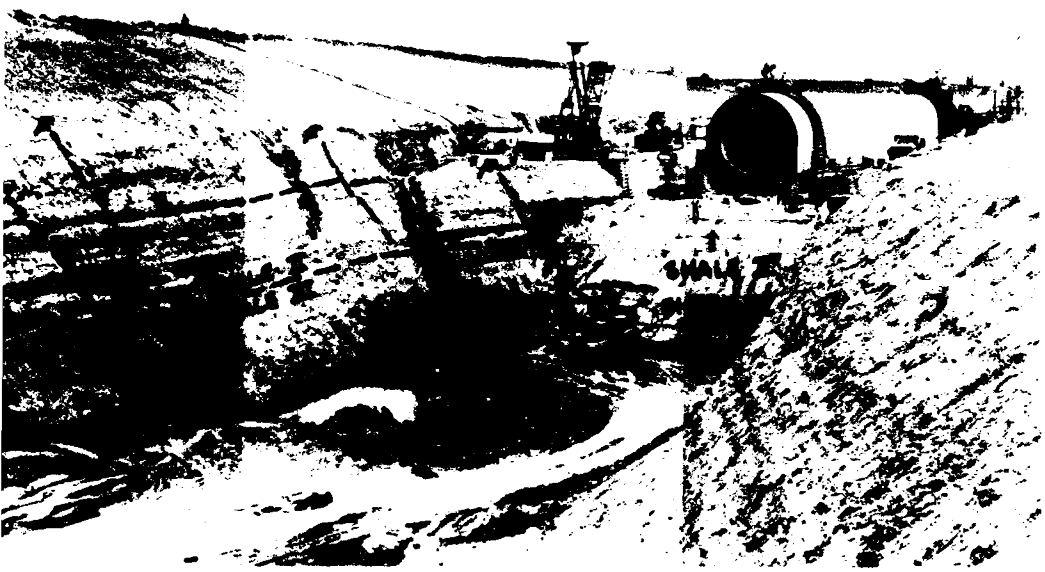


Figure 36. Outlet works, 2nd contract: View of faults in right (south) sideslope of stilling basin excavation.

Figure 37. Outlet works, 2nd contract: View of right (south) sideslope of the stilling basin and chute showing faults, lower depositional seam, and formation unit contacts.



works, 2nd contract: View of right (south) sideslope of
nd chute showing faults, lower depositional seam, and
itacts.

2

Figure 38. Outlet works, 2nd contract: View of one fault of Balcones system and two faults of local system in the right stilling basin sideslope.

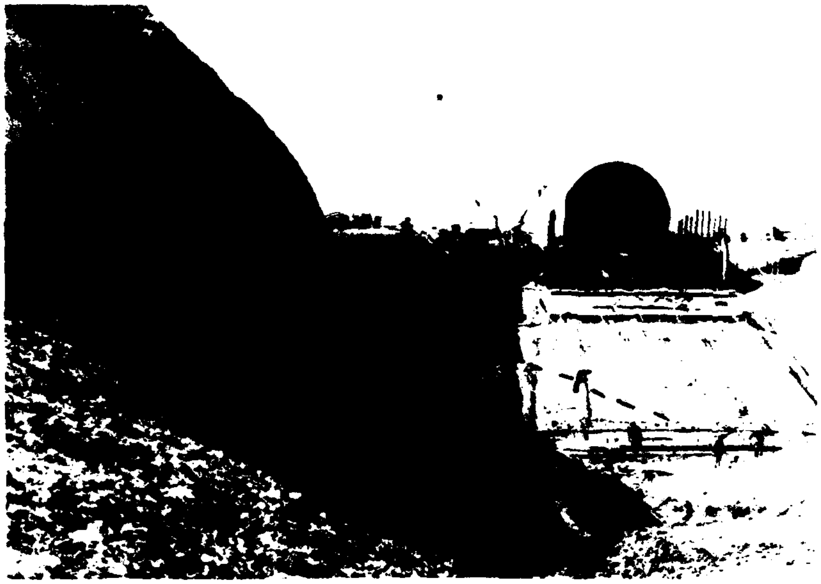


Figure 39. Outlet works, 2nd contract: Looking upstream along the right (south) stilling basin sideslope to the chute foundation and the downstream end of the conduit.

Figure 40. Outlet works, 2nd contract: View of lower, right chute slope, showing fault, lower depositional seam, and formational contact.

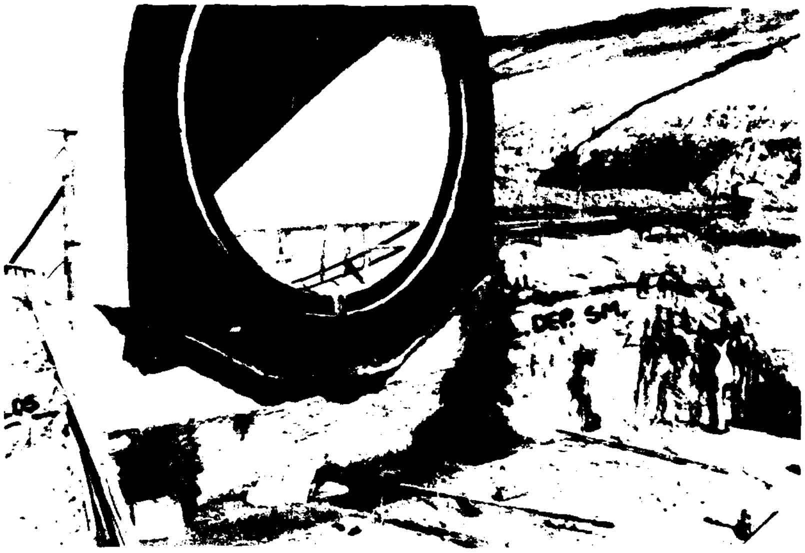


Figure 41. Outlet Works, 2nd contract: Downstream end of conduit and upstream end at chute foundation, showing lower depositional seam in sideslope of left manhole escavation.

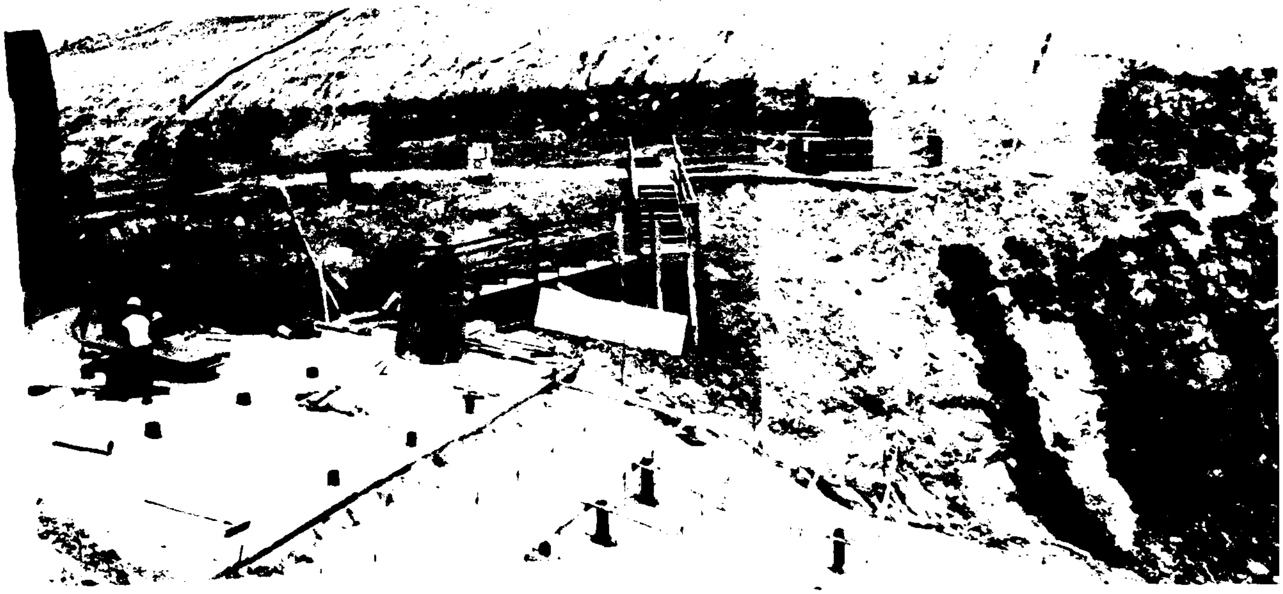


Figure 42.
and stilling

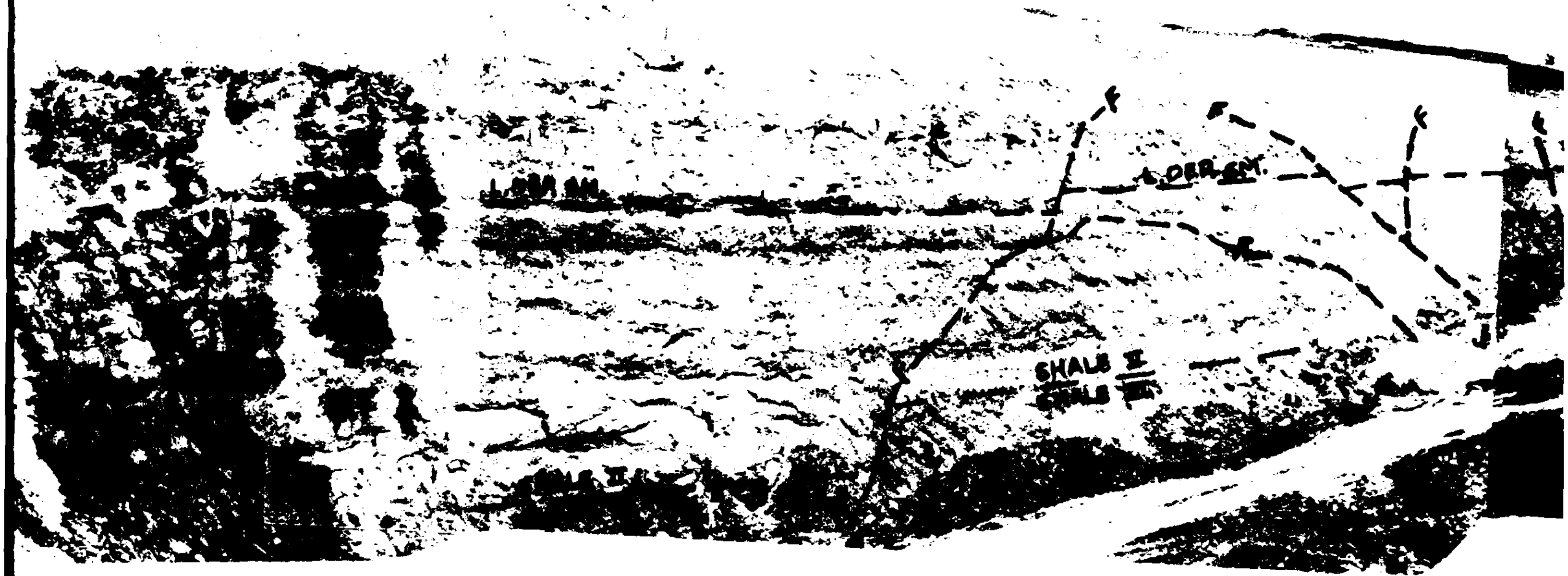
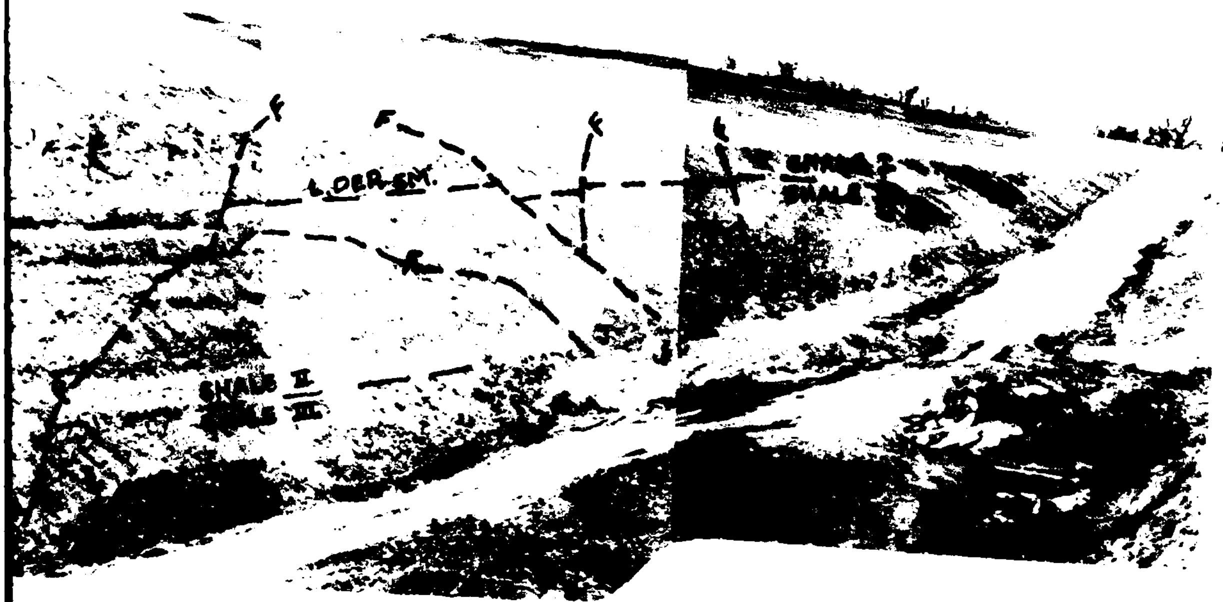


Figure 42. Outlet Works, 2nd contract: Left (north) sideslope of chute and stilling basin showing faults and the lower depositional seam.

2



of chute
seam.

3

Figure 43. Outlet Works, 2nd contract: Stilling basin, chute and headwall of conduit upstream from end sill under construction.

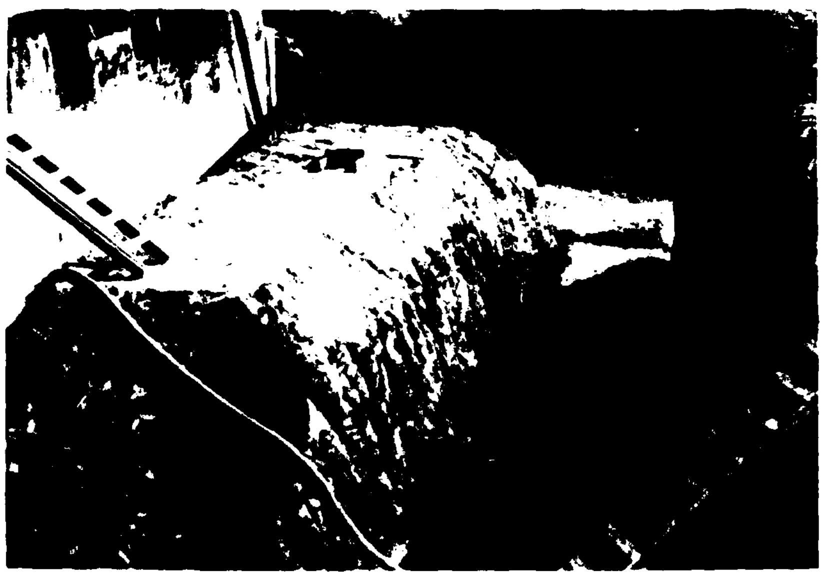


Figure 44. Outlet Works, 2nd contract: Shale block under end sill of stilling basin, looking right to left.

Figure 45. Outlet Works, 2nd contract: Looking upstream. Clean-up of shale "flat" for backfilling on right (south) side of conduit.

Figure 46. Outlet Works, 2nd contract: Looking downstream. Clean-up of shale "flat" for backfilling on right (south) side of conduit.

Figure 47. Outlet Works, 2nd contract: View of early stage of backfilling of outlet excavation, looking upstream along the centerline.

Figure 48. Outlet Works, 3d contract: View of additional backfilling outside right walls of chute and stilling basin. Note dark spall area in shale slope.

Figure 49. Outlet Works, 3d contract: View of spall area at right shale slope of figure 48.

Figure 50. Outlet Works, 3d contract: Detail view of spall area for scale and view of striations on surface.

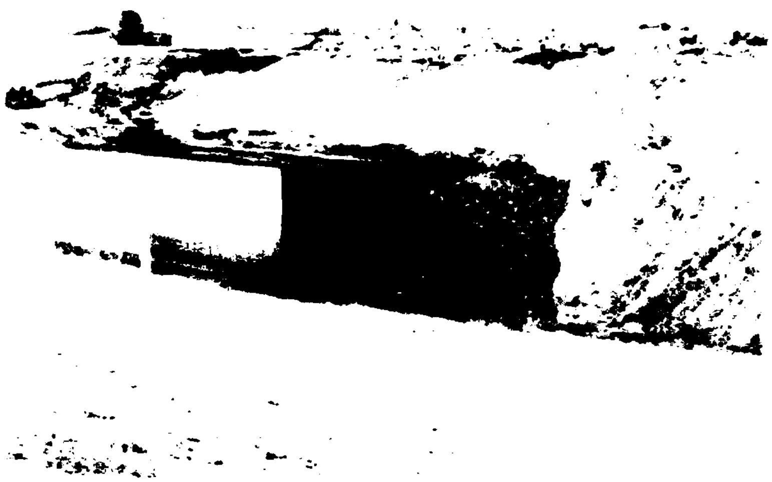
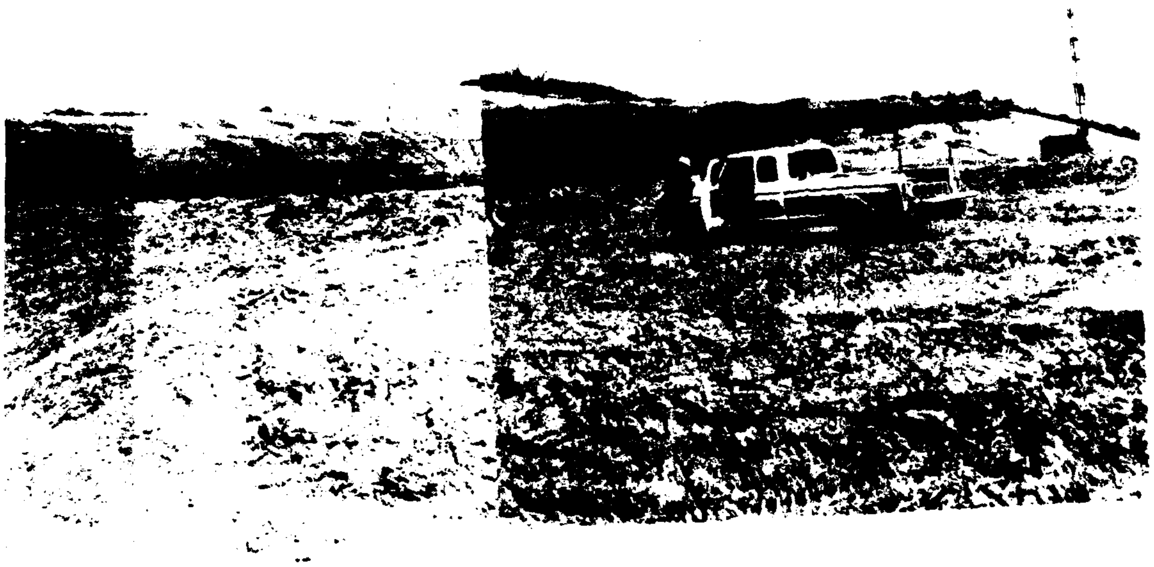


Figure 51. Outlet Works, 3d contract: Regrading of left sideslope outside left wall of stilling basin to correct the 2nd contract profile and to produce berm. Note shale cut immediately below berm. Compare view with that of *figure* 42.

Figure 52. Outlet Works, 3d contract: Close view of shale cut of figure 51. Note steeply dipping fault. Strike of fault here is parallel to outlet centerline.

Figure 53. Spillway, 3d contract: Upper portio
showing sectional view of cutoff trench back



ontract: Upper portion of left (north) sideslope
›f cutoff trench backfill (from 1st contract).

2

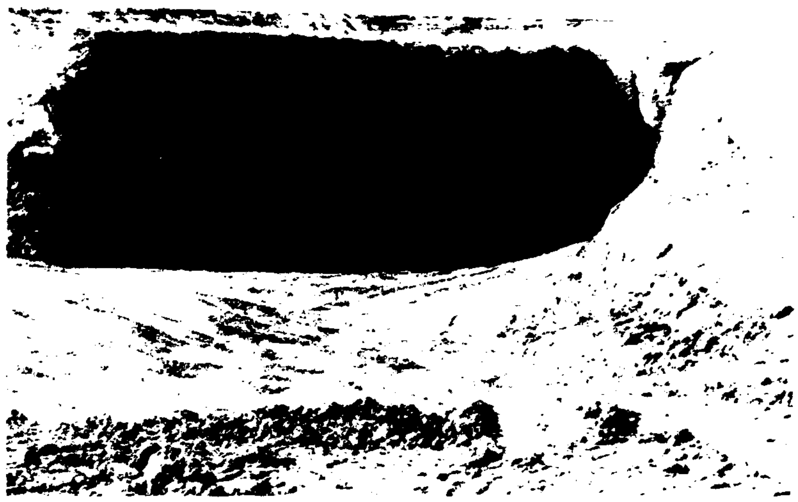


Figure 54. Spillway, 3d contract: View of excavation for left (north) wall refill. This location is just upstream from the site of figure 53, but view here is upstream and toward *centerline*.

Figure 55. Spillway, 3d contract: View of bottom of excavation for refill, looking left (north) and slightly upstream. The excavation for the left wall refill of figure 54 leaves this view to the left from upper center.

Figure 56. Spillway, 3d contract: View of shale bottom and backslope of right (south) half of refill excavation, looking toward right (south) side of spillway.

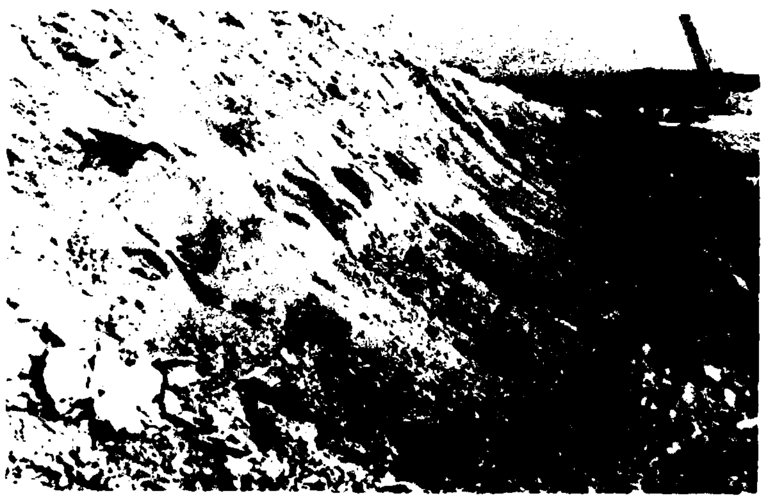


Figure 57. Spillway, 3d contract: Shale backslope of refill excavation in right (south) half of the spillway, looking toward the centerline. Note weathering pattern in the shale.

Figure 58. Spillway, 3d contract: Location of fault in backslope of refill excavation. Fault follows hand-dug steps in the weathered shale of the backslope to its contact with the overburden at the top of the slope. The fault is at station 28+62, 271 feet left at the top of the finished slope. The overburden shown was later removed in grading the approach.

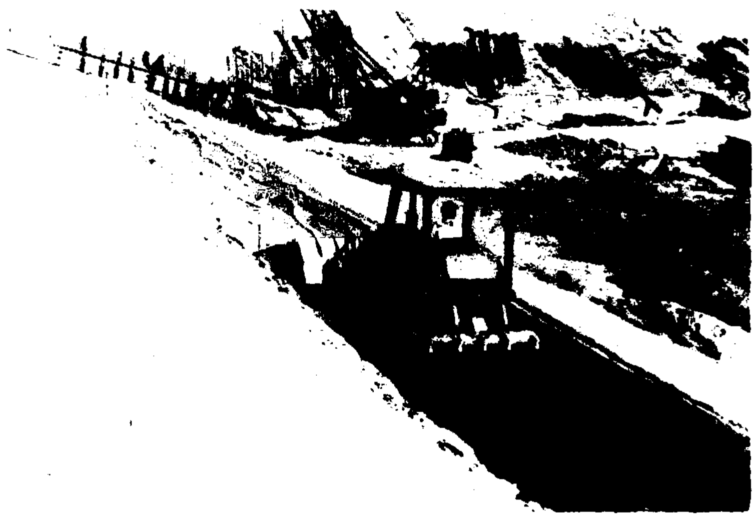


Figure 59. Spillway, 3d contract: Excavating for a chute cross drain by initial method (see text).

Figure 60. Spillway, 3d contract: Cross drain excavated by initial method ready for clean-up.

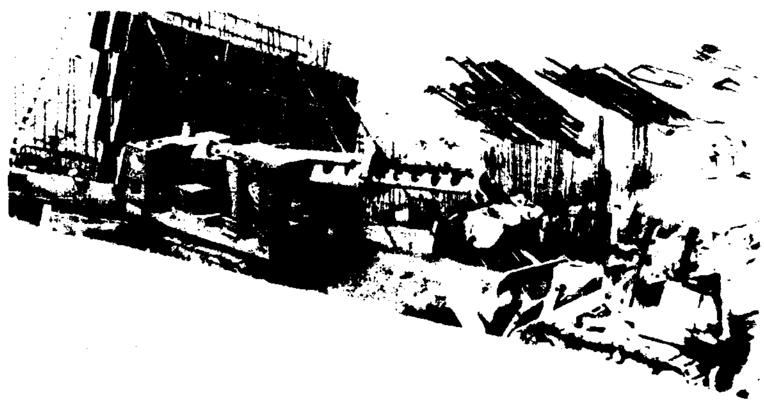


Figure 61. Spillway, 3d contract: Commencement of clean-up and machinery used under initial method of construction.

Figure 62. Spillway, 3d contract: Excavating a chute cross drain by final *method (see text).*

Figure 63. Spillway, 3d contract: Cross drain excavated by final method. Note large number of shale pull-outs in slope.

Figure 64. Spillway, 3d contract: Chute cross drain in area of no shale desiccation or structural disturbance. (just prior to covering with moist sand).

Figure 65. Spillway, 3d contract: Caterpillar backhoe bucket with smooth-mouth blade used to fine grade.

Figure 66. Spillway, 3d contract: Pull-outs caused by presence of two very thin, soft seams in the shale of the chute, which aided determination of fault displacement (see text).

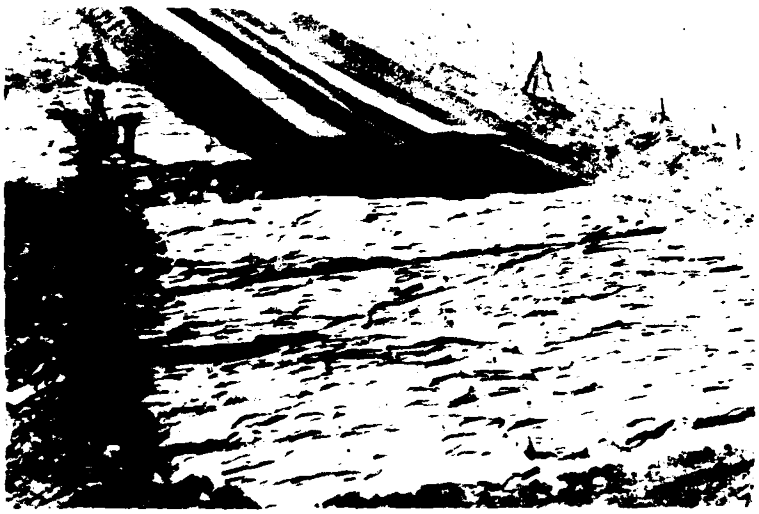


Figure 67. Spillway, 3d contract: Sanding the chute slopes and cross drains in strips.

Figure 68. Spillway, 3d contract: View across chute drainage blanket. Looking toward left wall.

Figure 69. Spillway, 3rd contract: Excavating and cleaning a strip of the chute foundation. View shows a fault trending from upper right to lower left.

Figure 70. Spillway, 3d contract: View diagonally across spillway chute (downstream and toward the right sideslope). View shows granular refill (near), the chute work slab with anchors, and the drainage blanket and fresh excavation in the distance.

Figure 71. Spillway, 3d contract: View showing the left (north) wall footings covered with protective concrete, the chute drainage blanket, and shale in the chute being covered with sand.

Figure 72. Spillway, 3d contract: Wire mesh being applied to shale of the outside slope of the right wall footing.

Figure 73. Spillway, 3d contract: Wire mesh attached to bridge nails in shale of outside slope of right wall footings.

Figure 74. Spillway, 3d contract: Pneumatic concrete applied to wire mesh over shale of the outside slope of the right wall footings. Commencement of the key along the outside of the wall footing is visible at the bottom of the photo.-

Figure 75. Spillway, 3d contract: View showing commencement of wall footing key with increased height to be covered with wire mesh and pneumatic concrete.

Figure 76. Spillway, 3d contract: View of upstream portion of the stilling basin foundation showing the effect of partings within the shale. Looking toward the centerline and left wall.

Figure 77. Spillway, 3d contract: View of downstream portion of the stilling basin foundation near the right (south) wall, showing abundant partings within the shale. Looking toward the right wall. There is a fault within the view where the parting dip abruptly diminishes.

Figure 78. Spillway, 3d contract: Crude oil bleeding from a shale parting in the backslope of the cross drain at the foot of the chute slope near the right wall.

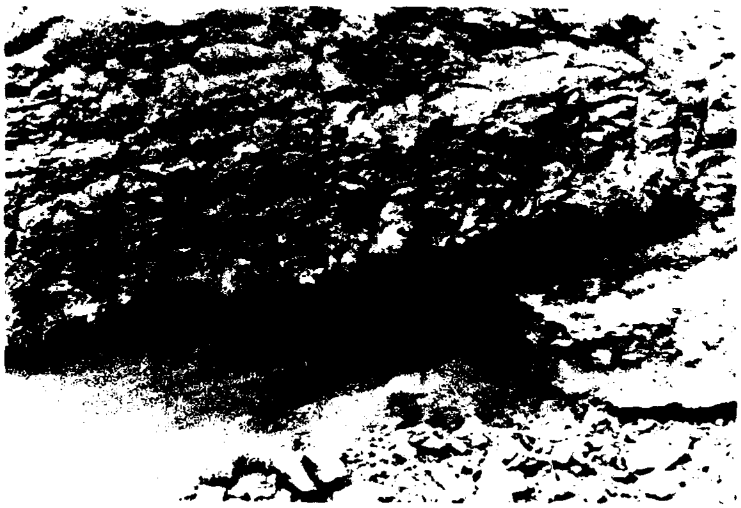


Figure 79. Spillway, 3d contract: Crude oil seep in the bottom of the cross drain at the base of the chute slope near the right wall.

Figure 80. Spillway, 3d contract: Shale parting, emitting crude oil in bottom of cross drain at the bottom of the chute slope near the right wall.

Figure 81. Spillway, 3d contract: Crude oil bleeding from shale in the bottom of the cross drain at the foot of the chute near the right wall

Figure 82. Spillway, 3d contract: View of slumped drainage blanket sand and resulting void under the work slab. Note: Water leaking from water-charged cross drainpipe. Site is next to left wall at foot of chute.

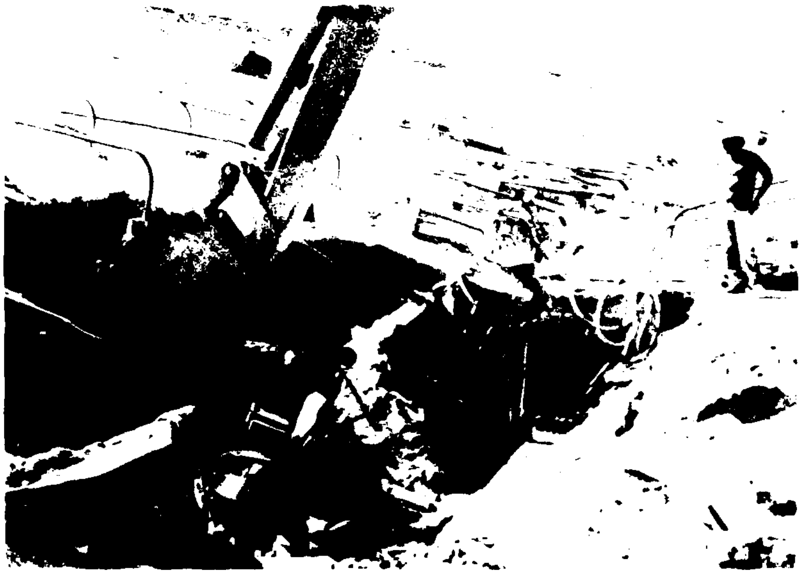


Figure 83. Spillway, 3d contract: Excavating manhole at offset 207.5 feet left at the base of the chute. Note void under the work slab both upstream and downstream from key concrete. Also key concrete reaching cross drainpipe.

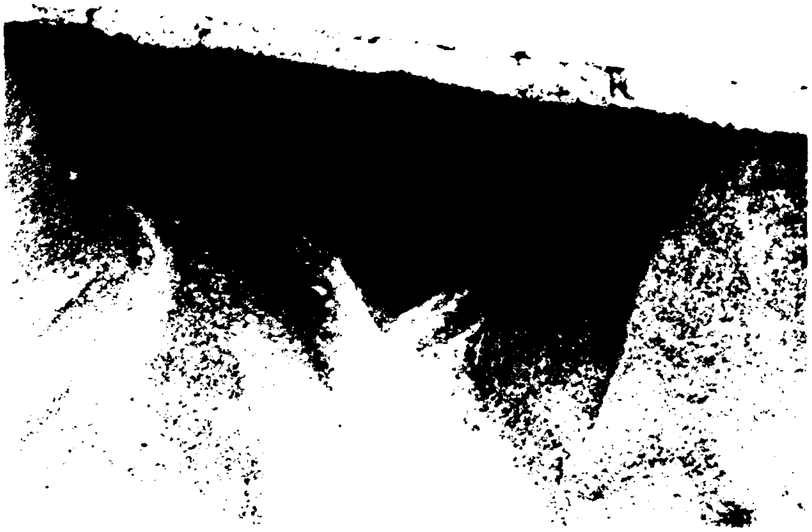


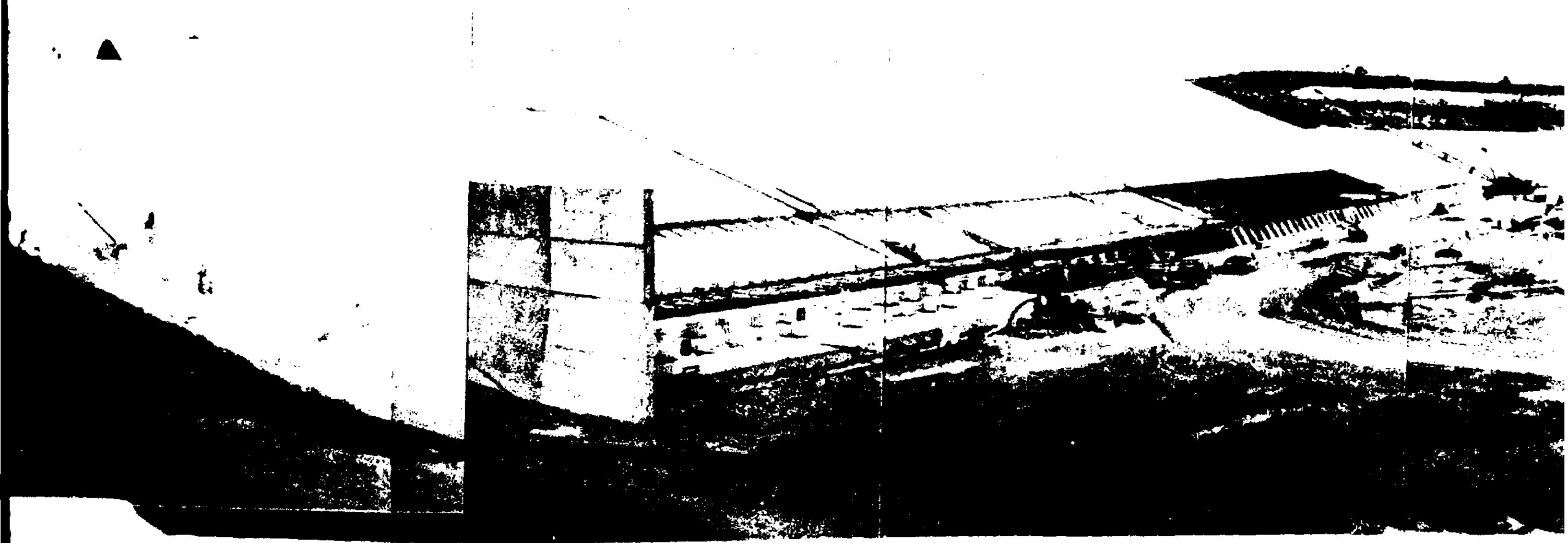
Figure 84. Spillway, 3d contract: Water flowing from charged cross drainpipe at the foot of the chute slope.

Figure 85. Spillway, 3d contract: Pumping from the 9 feet deep end sill key excavation. Looking toward left (north) wall. Note: drainage blanket does not extend this far downstream.

Figure 86. Spillway, 3d contract: View of section of partially flooded, 9 feet deep end sill key near the right (south) wall.

Figure 87. Spillway, 3d contract: Water flowing from partings in the shale approximately 4 feet below the top of the upstream slope of the end sill key.

Figu



e 88. Spillway, 3d contract: View of the nearly finished spillway.

2

spillway.

3

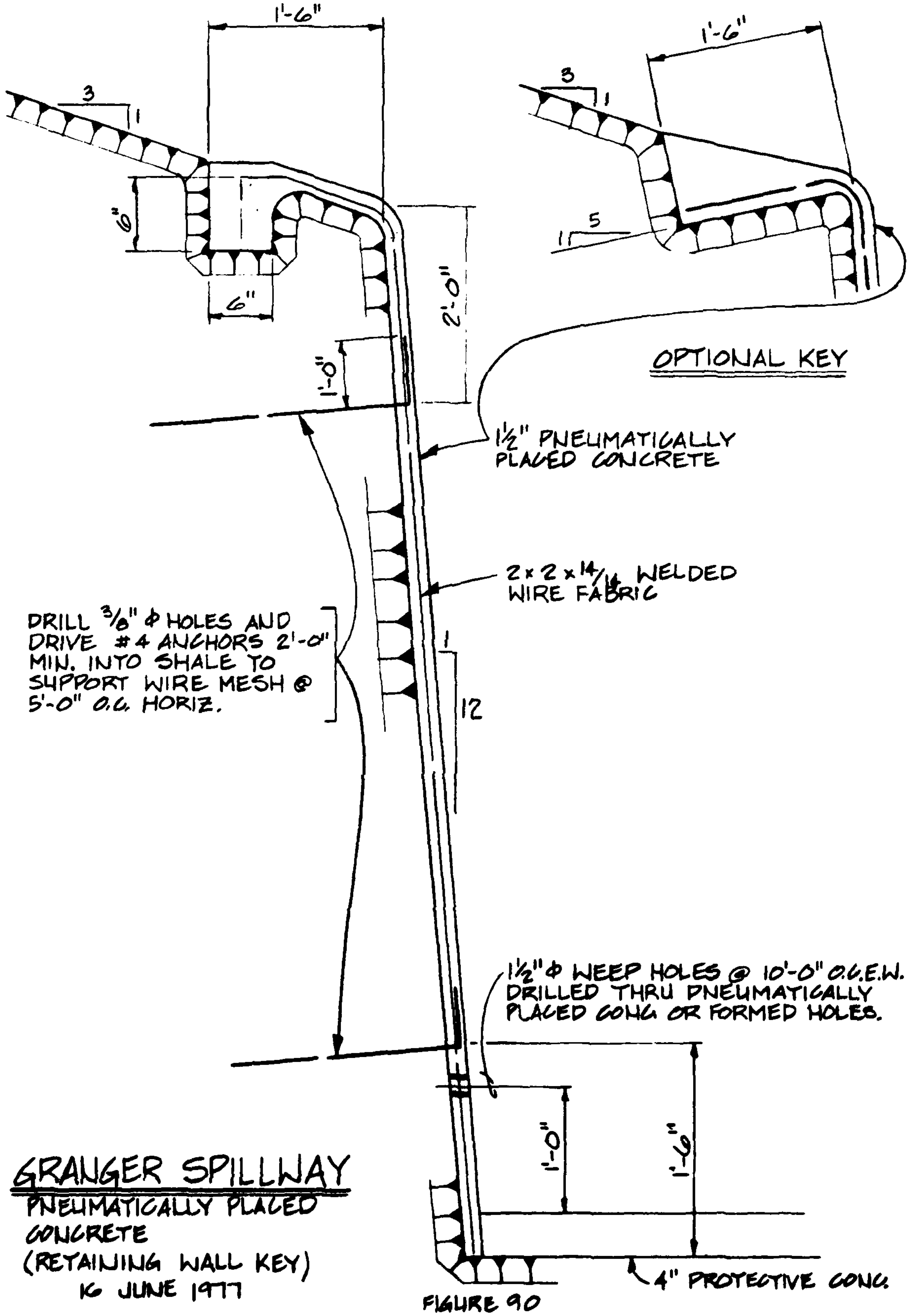
Figu
char

Figure 89. Spillway, 3d contract. View of the spillway, its discharge channel, and the embankment to the north (distant).

2

scharge

3



GRANGER SPILLWAY

PNEUMATICALLY PLACED CONCRETE (RETAINING WALL KEY)

16 JUNE 1977

FIGURE 90

END

DATE
FILMED
7 - 84

DTIC